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ACI
PROCEEDINGS
VOL. 55

82-Foot Prestressed 'Incor' Box Beams

—SPAN TROUBLED WATERS

● Among the flood control projects in the State of Pennsylvania is the Stillwater Reservoir that impounds waters of the Lackawanna River. Relocation of the highway in that area was part of the project, which included bridging the east branch of the river.

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SEPTEMBER 1958

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Vol. 30

PROCEEDINGS VOL. 55

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Coming next month in the JOURNAL

The October JOURNAL, dressed up for a memorable event in ACI history—the dedication of the new Institute headquarters building on October 29, 1958—has been designated the dedication issue. Three individual papers submitted by MINORU YAMASAKI, architect; CHARLES S. WHITNEY, structural engineer; and JOHN STRANG, contractor, recount some of the challenges in conception and execution of this unique ACI structure.

●

Also in the dedication issue, LYNDON WELCH discusses design and construction of a folded plate roof in "Folded Plate Dome Ideal for Auditorium." Design principles applied by FELIX J. SAMUELY in "Folded Slab Construction" include some of his work on folded slab roofs, stair landings, and galleries, with special attention to shear forces at folds, and to stresses produced by differential deformation in the slabs.

●

"R/C Core Main Structural Element in 22-story Office Tower," by OTTO SAFIR, describes a structure built for the British Columbia Electric Co. at Vancouver.

J. R. GASTON and EIVIND HOGNESTAD present their joint paper on "Precast Concrete Girders Reinforced with High Strength Deformed Bars."

●

A trio of well-known authors, A. C. SCORDELIS, T. Y. LIN, and H. R. MAY, presents "Shearing Strength of Prestressed Concrete Lift Slabs."

"Origin, Evolution, and Effects of the Air Void System in Concrete. Part 4—The Air Void System in Job Concrete" concludes a series of papers prepared by RICHARD C. MIELENZ, VLADIMIR E. WOLKODOFF, JAMES E. BACKSTROM, and RICHARD W. BURROWS.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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Proposed

Recommended Practice for Selecting Proportions for Structural Lightweight Concrete*

Reported by Subcommittee on Proportioning Lightweight Aggregate Concrete, ACI Committee 613

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SYNOPSIS

This subcommittee report is intended as a supplement to ACI Standard "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" and describes a procedure for proportioning structural grade concrete containing lightweight aggregates. This procedure does not require the use of values for specific gravity or absorption of the aggregates but utilizes a "specific gravity factor." Use of this factor is illustrated and examples are included for proportioning both air-entrained and non-air-entrained mixes.

CHAPTER 1—INTRODUCTION

101—Scope and limits

(*a*) Under this title the subcommittee has limited its discussion and recommendations to structural grade lightweight aggregate concrete. Structural lightweight concrete is defined as having a 28-day compressive strength in excess of 2000 psi and an air dry unit weight less than 115 lb per cu ft. Many expanded shales, slates, clays, slags, and some natural aggregates, such as scoria, are used in this class of concrete. Proportioning of very lightweight concrete used primarily for insulation is not included.

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This report was prepared as a part of the work of ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes. The report was submitted to the main committee (26 members) with 17 voting affirmatively and none negatively. It is released by the Standards Committee for publication and discussion with view to its consideration for adoption as an Institute Standard at the 55th annual convention, Los Angeles, Calif., Feb. 23-27, 1959.

†Deceased.

(*b*) The principles of normal weight concrete proportioning as established by ACI Standard "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)" apply directly to lightweight aggregate concretes, but generally the application of these principles is difficult. However, conventional procedures may be used with good results for such lightweight aggregates as are generally characterized by rounded particle shape, coated or sealed surfaces, and relatively low values of absorption.

CHAPTER 2—FACTORS AFFECTING PROPORTIONS OF LIGHTWEIGHT CONCRETES

201—General

(*a*) Some lightweight aggregates have absorption values of more than 12 percent and so may have more than 200 lb of absorbed water per cu yd of concrete. The question of how much water is absorbed and how much water actually occupies space in the concrete is the principal difficulty in proportioning by absolute volume procedures, when applied to this class of lightweight aggregates. The high values of absorption and the fact that the absorption may continue at an appreciable rate for several days make it difficult to determine correct values of absorption and specific gravity of the aggregate. Because of these complications the established relationships cannot be applied with the same confidence as for normal weight aggregates.

(*b*) The net water-cement ratio of most lightweight aggregate concretes can *not* be established with sufficient accuracy for use as a basis of mix proportioning. Lightweight aggregate concrete mixes are established by a series of trial mixes proportioned on a cement content basis at the required consistency.

202—Aggregate

(*a*) Several investigators[1,2,3,4] have determined that the moisture content of the aggregate immediately prior to mixing has little effect on the compressive strength of the concrete. Whether the aggregates are dry or have free surface moisture makes little difference as long as the moisture content is known and accounted for at the time the aggregates are batched. Dry aggregates produce concrete of lower unit weights than wet aggregates and allow easier control of batch quantities. Concrete made with saturated aggregate will be more vulnerable to damage by freezing and thawing than concrete made with dry aggregate. However, it is desirable that the aggregates be damp at the time of mixing to reduce the amount of water absorbed from the mix and so reduce the rate of loss of slump. Damp materials also show less tendency to segregate during storage and transportation than dry materials. When the aggregate is in a less than saturated condition, it is desirable to mix the aggregate with about two-thirds of the required mixing water for a short period prior to introduction of cement and air-entraining admixture. This reduces the effective loss of cement into the vesicles in the surface of porous aggregates.

(*b*) The grading of the fine and coarse aggregate and the proportions used have an important effect on the concrete. A well graded aggregate will have a minimum void content and so will require a minimum amount of paste to fill these voids. This will result in the most economical use of cement and will provide maximum strength with minimum volume change due to drying shrinkage and temperature changes.

(*c*) For sand-and-gravel aggregates the bulk specific gravities of materials retained on the different sieve sizes are nearly equal, so the fineness modulus, on a weight basis, gives a true indication of the volumes occupied by each size of material. However, the bulk specific gravity of the various size fractions of lightweight aggregate increases as the particle size diminishes. Some coarse aggregate particles may float in water while the particles finer than the No. 100 sieve have a specific gravity approaching that of sand and gravel. It is the *volume* occupied by each size fraction and *not the weight* of material retained on each sieve that determines the final void content, paste content, and workability of the concrete. The fineness modulus by weight and by volume for a lightweight aggregate are computed in the following example.

Sieve size	Percent retained, by weight	Cumulative percent retained, by weight	Specific gravity	Percent retained, by volume	Cumulative percent retained, by volume
4	0	—	1.40	—	—
8	21.6	21.6	1.55	25.9	25.9
16	24.4	46.0	1.78	25.4	51.3
30	18.9	64.9	1.90	18.5	69.8
50	14.0	78.9	2.01	13.0	82.8
100	11.6	90.5	2.16	9.9	92.7
Pan	9.5	100.0	2.40	7.3	100.0

Fineness modulus (by weight) = 3.02 Fineness modulus (by volume) = 3.23

(*d*) The fineness modulus (by volume) of 3.23 indicates a considerably coarser grading than that normally associated with the fineness modulus of 3.02 by weight. From data presented by Richart and Jensen[1] on another lightweight aggregate, the fineness modulus of 2.83 on a weight basis converts to 3.21 on a volume basis. Therefore, lightweight aggregates require a larger percentage of material retained on the finer sieve sizes, on a weight basis, than do the heavier aggregates to provide an equal void content. Grading and other properties of lightweight aggregate for structural concrete are specified in ASTM C 330.

203—Air entrainment

(*a*) Air entrainment is desirable in most concrete. It improves workability and resistance to weathering, and it decreases bleeding and obscures grading deficiencies. Strength is generally reduced, but the reduction is not great with normal air contents, particularly when cement content is maintained, and

becomes important only with richer mixes. The advantages of air entrainment are most apparent in lean or otherwise harsh mixes where strengths may actually be increased. Most lightweight aggregate concretes will contain from 2 to 4 percent entrapped air, but this has negligible effect on workability and durability and additional entrained air is desirable. The durability of lightweight aggregate concretes has generally been considered to be very good even without air entrainment.[2,6] However, where air entrainment is required to provide protection against freezing and thawing, it is recommended that all lightweight aggregate concretes contain not less than 6 percent total (entrapped plus entrained) air. T. C. Powers[7] and others have shown that the spacing of the air bubbles is more important than the total quantity of air entrained. But since this factor cannot be readily determined in the field, adequate weathering protection can only be assured by the use of recognized air-entraining admixtures and by obtaining the recommended quantities of entrained air.

(*b*) There are several methods of determining the amount of entrained air. It is indicated by the difference in unit weights of similar concrete mixes with and without an air-entraining admixture and with the water adjusted to provide a given slump. It can be computed from these mixes since the difference in volumes of the two batches plus the volume difference in water requirement equals the volume of purposely entrained air. It can also be computed by use of the "specific gravity factor" which is discussed later in this report. The pressure type meter can be used if it is properly calibrated, although with many air-entrained lightweight concretes it must be used at reduced pressure. The volumetric method described in ASTM C 173-55T gives the most reliable results and this method is recommended. It has the advantage that the air content of both air-entrained and non-air-entrained concrete can be measured.

CHAPTER 3—ESTIMATING PROPORTIONS

301—Proportions of fine and coarse aggregate

(*a*) A good estimate of the proportions of fine and coarse aggregate may be obtained by the maximum density method. The proportions are adjusted to produce a maximum unit weight of the combined aggregate. Due to the difference in the specific gravities of the fine and coarse fractions the curve may not show a maximum at a point below 100 percent of the fine fraction; however, a rather sharp change in curvature will be observed. This change in curvature is indicative of the region of minimum void content, and can be verified by computation if values of the apparent specific gravities of the coarse and fine aggregate fractions are known. Results of such procedure are illustrated in Fig. 301(a).

(*b*) This procedure will generally indicate that, for ¾-in. maximum size aggregate, material passing the No. 4 sieve should be between 40 and 60 percent

of the total aggregate, based on dry loose volume. As the maximum size of aggregate decreases the proportion of fine aggregate increases. With air entrainment and richer mixes a lower percentage of fines is normally used. The void content changes only slightly between limits of 40 and 60 percent. The amount of fine aggregate is generally kept as low as possible while still providing a margin of safety for good workability. However, some producers of lightweight aggregate concrete have found that maximum strengths are not obtained by using the largest quantity of coarse aggregate. The relatively weak physical structure of the large size aggregate reduces the concrete strength if an excessive amount of coarse aggregate is used.

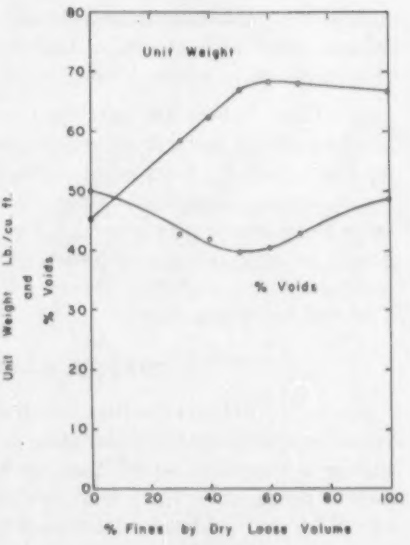

Fig. 301(a)

302—Estimate of cement and water requirements

(*a*) Some lightweight aggregates are rounded with relatively smooth surfaces, others are extremely angular with very rough surfaces. These characteristics account for the wide range in amount of net water required to produce a concrete of a given consistency with different *saturated* aggregates. The *net* water required to produce a 2-in. slump in non-air-entrained concrete averages about 390 lb per cu yd of concrete. However, amounts varying from 300 to 450 lb per cu yd are encountered with different aggregates. This wide range in water requirement is reflected in a corresponding range of cement contents to produce a given strength. The structural strength of different aggregates also has an important effect on the cement requirement, particularly for higher strength concretes.

(*b*) Producers of the various lightweight aggregates have devoted considerable time to studies of their particular aggregates. Their recommendations provide the best estimate of the required cement content and other mix proportions and should be given first consideration. Because of the wide range of cement contents required to produce concrete of the same compressive strength with the various lightweight aggregates, the subcom-

TABLE 302(b)—APPROXIMATE RELATIONSHIP BETWEEN STRENGTH AND CEMENT CONTENT

Compressive strength, psi	Cement content bags per cu yd*
2000	4 to 7
3000	5 to 8
4000	6 to 9
5000	7 to 10

*U. S. standard bag of cement weighing 94 lb. The Canadian standard bag of cement weighs 87.5 lb.

mittee was reluctant to provide any indication of the compressive strength to be associated with a certain cement factor. However, Table 302(b) provides this relationship within broad limits.

(*c*) These values are suggested only as a guide in proportioning the first trial mixes. A number of references to published data which might also be used as a guide are included at the end of this report. As with heavier concretes, the minimum amount of mixing water should be used that will allow the concrete to be adequately placed. The use of wetter mixes increases the opportunity for segregation and lowers the bond and compressive strengths and the resistance to weathering action. Under most conditions, slumps less than 4 in. will be satisfactory.

CHAPTER 4—TRIAL MIXES

(*a*) Due to the difficulties involved in the determination of a satisfactory value for specific gravity and absorption of the aggregate, a method of proportioning is suggested which does not require the use of these values. The first step in preparation for a trial mix is to determine the dry loose unit weights (ASTM C 330) and moisture contents of the fine and coarse aggregate fractions. Trial mixes are then made with at least three different cement contents. The first mix is made with estimated quantities of the various materials and the water is added to produce the required slump. The quantities for this first mix might be estimated in the following manner:

Assume that the mix is to contain 6 bags of cement per cu yd, and that the dry loose unit weights for the fine and coarse aggregates are 56 and 45 lb per cu ft, respectively. A slump of about 2 in. is required. About 32 cu ft of dry loose aggregate (sum of uncombined fine and coarse volumes) are required to produce a cubic yard of concrete. Most lightweight aggregates will require from 31 to 33 cu ft, but values as low as 28 cu ft may occur with particularly well graded and rounded aggregates. If the aggregate is produced in two sizes, fine and coarse, these are proportioned in equal volumes. Water is added to produce the required slump. A trial batch of approximately 1 cu ft would then require:

Cement	$\frac{6 \times 94}{27}$	= 20.9 lb
Fine aggregate	$\frac{16 \times 56}{27}$	= 33.2 lb
Coarse aggregate	$\frac{16 \times 45}{27}$	= 26.7 lb
Water	$\frac{480}{27}$ (includes water absorbed during mixing)	= 17.8 lb
	Total weight	98.6 lb

The wet unit weight is 97.0 lb per cu ft and the air content test indicates 2.5 percent entrapped air. The yield 98.6/97 = 1.016 cu ft. Quantities per cubic yard of concrete are obtained by multiplying the batch quantities by the ratio of cubic feet in a yard to cubic foot of batch 27/1.016 = 26.6.

Cement	556 lb per cu yd	= 5.91 bags per cu yd
Fine aggregate	882 lb per cu yd	
Coarse aggregate	710 lb per cu yd	
Water	473 lb per cu yd	

(*b*) Additional trial mixes could be made by estimating cement and aggregate quantities and establishing the yield, cement and water content, and other fundamental relationships. However, by the use of a "specific gravity factor"[11] for the aggregate, all other trial mixes with this aggregate can be proportioned with considerable confidence. The specific gravity factor is obtained in the following manner:

$$\text{Solid volume of cement} \qquad \frac{556}{62.4 \times 3.15} = 2.83 \text{ cu ft}$$

$$\text{Volume of water} \qquad \frac{473}{62.4} = 7.58 \text{ cu ft}$$

$$\text{Volume of entrapped air} \qquad 2.5 \text{ percent} = \frac{0.67 \text{ cu ft}}{11.08 \text{ cu ft}}$$

The aggregate then occupies $27 - 11.08 = 15.92$ cu ft. The coarse and fine aggregate were used in equal parts by volume, so they each occupy 7.96 cu ft. This is not precisely correct, but the error becomes unimportant when the specific gravity factor method is used. The specific gravity factor expresses the relationship between the dry weight of the aggregate and the space it occupies, assuming that no water is absorbed during mixing.

$$\text{Specific gravity factor, fine aggregate} \qquad \frac{882}{62.4 \times 7.96} = 1.78$$

$$\text{Specific gravity factor, coarse aggregate} \qquad \frac{710}{62.4 \times 7.96} = 1.43$$

(*c*) This is only a *factor* and not a specific gravity value defined by ASTM because the method does not account for any water absorbed during mixing. However, in subsequent mixes with this aggregate in the same moisture condition, the volume of water absorbed during mixing is nearly constant. The specific gravity factor can then be used as though it were the apparent specific gravity and additional mixes can be proportioned by a procedure similar to the absolute volume method outlined in ACI 613-54.

(*d*) The first trial batch may appear harsh and need a higher percentage of fine materials. Additional trial mixes are made using less coarse aggregate until the desired workability is obtained. These and other mixes can be proportioned by the method outlined in the following example.

Assume the 5.91-bag trial batch is judged to have the desired workability and no further adjustments are required. We now want to proportion an 8-bag mix at the same consistency. *The quantity of coarse aggregate is maintained constant. The water requirement also is considered to remain constant.*

Solid volume of cement	$= \frac{752}{62.4 \times 3.15}$	= 3.83 cu ft
Volume of water	$= \frac{473}{62.4}$	= 7.58 cu ft
Solid volume of coarse aggregate	$= \frac{710}{62.4 \times 1.43}$	= 7.96 cu ft
Air	2.5 percent	= 0.67 cu ft
Total volume of ingredients except fine aggregate		= 20.04 cu ft
Volume of fine aggregate = 27 − 20.04		= 6.96 cu ft
Weight of fine aggregate = 6.96 × 62.4 × 1.78		= 773 lb

A trial mix is made using these proportions although the quantity of water may require minor adjustment. The unit weight and air measurement are taken and the actual quantities of material per cubic yard are computed.

(*e*) Air-entrained concrete mixes are proportioned in the same manner. However, the water content is generally reduced 2 to 3 percent for each 1 percent of entrained air. For example, if we want to repeat the 5.91-bag mix but include 4 percent purposely entrained air:

Solid volume of cement	$= \frac{556}{62.4 \times 3.15}$	= 2.83 cu ft
Volume of water	$= \frac{426}{62.4}$	= 6.83 cu ft
Solid volume of coarse aggregate	$= \frac{710}{62.4 \times 1.43}$	= 7.96 cu ft
Volume of air (2.5 percent + 4 percent) = 0.065 × 27		= 1.76 cu ft
		19.38 cu ft
Volume of fine aggregate = 27 − 19.38		= 7.62 cu ft
Weight of fine aggregate = 7.62 × 62.4 × 1.78		= 846 lb

(*f*) Additional trial mixes should be made which include at least three cement contents both with and without air entrainment. Graphs may be prepared relating the cement content and unit weight to the 28-day compressive strengths for use in proportioning mixes at other strength levels.

JOB CONTROL

(*a*) Job control of lightweight aggregate concrete is not within the scope of this subcommittee, but to make effective use of this proposed recommended practice the following fundamentals for lightweight concrete control are suggested.

(*b*) Job control is obtained by keeping constant the cement content, slump, and *volume of dry aggregate* per cubic yard of concrete regardless of variations in absorbed or surface moisture. If the density of the aggregate does not vary, constant *volume* can be maintained by keeping the *dry weight* of aggregate constant. *Unit weight* of the *fresh concrete* should be determined at *frequent* intervals. *A change in unit weight indicates a change in air content or change in weight of aggregate.* An air content determination will establish whether or not the correct amount of air is entrained. If the weight of aggregate has changed, it is due to a change in moisture, a change in gradation, or a change in density of the aggregate. Checks of moisture content, gradation, and unit weight can be easily made, and these will reveal the cause of the change. If the *moisture* content has changed, the aggregate weight is adjusted to keep the *weight of dry aggregate* constant. If the *density* of the aggregate has changed, the weight of dry aggregate is adjusted to keep the *volume of dry aggregate* constant.

(*c*) It must be remembered that the principle of the water-cement ratio law applies to lightweight aggregate concrete. When mixes are proportioned on a cement content basis, the compressive strength associated with a certain cement content is obtained only at a given consistency. If it is found necessary to increase the slump, the increase in water content should be accompanied by a proportionate increase in cement content.

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10. Shideler, J. J., "Lightweight-Aggregate Concrete for Structural Use," ACI JOURNAL, Oct. 1957, *Proc.* V. 54, pp. 299-328.

11. Nelson, G. H. and Frei, O. C. "Proportioning and Control of Lightweight Structural Concrete," ACI JOURNAL, Jan. 1958, *Proc.* V 54, pp. 605-622.

12. The following standards and tentatives of the American Society for Testing Materials are particularly applicable to structural lightweight concrete:

ASTM C 330 Tentative Specifications for Lightweight Aggregates for Structural Concrete

ASTM C 136 Method of Test for Sieve Analysis of Fine and Coarse Aggregates (limits in C 330)

ASTM C 29 Tentative Method of Test for Unit Weight of Aggregate (shoveling procedure)

ASTM C 173 Tentative Method of Test for Air Content of Freshly Mixed Concrete by the Volumetric Method

Discussion of this report should reach ACI headquarters in triplicate by Dec. 1, 1958, for publication in the March 1959 JOURNAL.

Design of Concrete Overlays for Pavements*

Reported by Subcommittee VIII, ACI Committee 325

L. M. ARMS
Chairman

HENRY A. AARON

L. A. PALMER

SYNOPSIS

Report traces the development of empirical formulae for structural capacity of two slabs, one superimposed on the other, and gives equations developed by Marcus and Palmer which permit determination of stresses in both the upper and lower slabs. Locating joints in the overlay above joints in the old slab is recommended. Design of reinforcement for the overlay is considered, as well as combining resurfacing with pavement widening. Also discussed are: relative merits of a separation course between old and new slab; bonded resurfacing; and effect of subgrade condition.

Concrete overlays, otherwise called concrete resurfacings, have an experience record of widespread use in the United States. Resurfaced pavements now in service cover a period of not less than 43 years. During 1957 a survey of all known street and highway concrete resurfacing projects included over 300 projects in 28 states. Of these, 40 percent were rated as in excellent condition, 40 percent as good, 18 percent as fair and 2 percent as poor. The "poor" ratings applied to pavements of greater ages. Experience on these projects has supplied the information on which most resurfacings have been designed up to this time.

OVERLAY SLAB THICKNESS

Until recently there has been no comprehensive analysis of stresses involved in multiple layers of concrete pavements similar to the Westergaard and Pickett studies of stresses in a single slab. It is not known by whom† or when the suggestion was first made for use of the formula which assumed that the structural capacity of two slabs, one superimposed on the other, is equivalent to that of a single slab the square of whose thickness is equal to the

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Prepared as a part of the work of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports. The report was submitted to letter ballot of the main committee which consists of 15 members; 12 members returned their ballots, of whom 12 have voted affirmatively, and none negatively.

†In a paper "Highway Research in Illinois," published in *Transactions*, ASCE, V. 87, 1924, p. 1209, Fig. 17. Clifford Older, chief highway engineer for Illinois, used a similar formula for evaluating monolithic brick sections of the Bates Test Road. It must be noted that these were not concrete slabs resurfaced with brick, but were built in such a way that there was a substantial bond between the concrete and the brick layers. In other words, an attempt was made to produce a monolith, hence the term "monolithic brick." The load carrying capacity was considerably less than that of a single slab having thickness equal to the total thickness of concrete and brick, but was about that of a single slab having an "equivalent thickness" computed by this formula. This may well have been the original use of the formula.

sum of the squares of the two slabs. Expressed mathematically this formula becomes:

$$T^2 = T_O^2 + T_R^2 \quad \ldots\ldots (1)$$

in which T_O = thickness of old or original concrete slab which is overlaid by a new slab having thickness T_R; T_R = thickness of resurfacing slab laid on old slab; and T = thickness of a single slab having structural capacity equivalent to that of the slabs of thickness T_O and T_R.

This formula came into use with the full understanding that it was not technically accurate. It may be approximately correct under the conditions that (1) the two slabs have the same stiffness, and (2) that there is no friction between them. Since it is most unlikely that the two slabs will be of the same stiffness, it is to be expected that one will be stressed more than the other. This is offset by the fact that, normally, considerable friction will exist between the two slabs which will cause them to act to some degree as an integral unit and thus reduce the stresses below what they would be if the two acted separately with no friction between them.

In applying the formula to resurfacing design, the conditions are further complicated by the properties of the concrete in the old and new slabs. The old pavement may be fatigued by long continued overloads or have suffered other deterioration through a normal lifetime of service. To correct for this factor, it has been common practice to introduce a coefficient C to evaluate the condition of the old slab. The formula then becomes:

$$T^2 = C\,T_O^2 + T_R^2 \quad \ldots\ldots (2)$$

in which C = coefficient for evaluation of old concrete slab. Suggested values for C are as follows:

$C = 1$ if old pavement is in good condition
$C = 0.75$ if old pavement has a few corner breaks but no progressive breakage
$C = 0.35$ if old pavement is badly cracked or shattered

Following a series of load tests on resurfacing slabs at Lockbourne Air Force Base in Ohio in 1947 and 1948, the Ohio River Division Laboratory of the Corps of Engineers proposed a further modification of the formula which recognized the fact that friction between the two slabs caused the system to have somewhat greater capacity than indicated by Eq. (1) or (2). The modified formula* took this form:

$$T^{1.87} = C\,T_O^2 + T_R^2 \quad \ldots\ldots (3)$$

*Since this paper was written, a new Corps of Engineers manual on engineering and design of rigid airfield pavements has been released in which this formula is modified as follows:

$$T^{1.4} = CT_O^{1.4} + T_R^{1.4} \quad \ldots\ldots (3a)$$

or

$$T_R = \sqrt[1.4]{T^{1.4} - CT_O^{1.4}} \quad \ldots\ldots (3a)$$

It appears that Eq. (3a) calls for resurfacing thickness slightly greater than Eq. (3) but less than required by Eq. 2.

This latter formula has come into use for the design of resurfacing slabs resting directly on the old pavement so that a considerable friction exists between the two. Some bond may develop between the old and new but this equation does not call for the two slabs to be actually bonded. For pavements with a separating course which reduces or eliminates friction between the two slabs it is common practice to utilize Eq. (2). Transposed to solve for the resurfacing thickness the two equations take these forms.

$$T_R = \sqrt{T^2 - C\,T_O^2} \quad \ldots\ldots (2)$$

$$T_R = \sqrt{T^{1.87} - C\,T_O^2} \quad \ldots\ldots (3)$$

These formulas are recognized as empirical. A design procedure based on rational analyses of stresses is to be preferred. More than one agency is known to be working on the subject.

Development by Marcus and Palmer

Formulas credited to Henri Marcus of the Bureau of Yards and Docks of the U. S. Navy were presented in August, 1955, in a progress report of the American Society of Civil Engineers committee on the design of overlay pavements. The report appears as Paper 777, *Proceedings*, ASCE, V. 81. The same formulas were also derived by L. A. Palmer in a paper, "Design of a Concrete Overlay on Old Concrete Pavement," in the U. S. Navy's *Budocks Technical Digest*, No. 79, for August, 1957. The formulas developed by Marcus and Palmer follow:

$$f_1 = f_0 \frac{E_1 h_1^3}{E_1 h_1^3 + E_2 h_2^3} \quad \ldots\ldots (4)$$

$$f_2 = f_3 \frac{E_2 h_2^3}{E_1 h_1^3 + E_2 h_2^3} \quad \ldots\ldots (5)$$

where

f_1 = maximum tensile stress in the upper (new) slab under a given applied wheel load when the new slab rests on the underlying old slab

f_0 = maximum tensile stress in the upper (new) slab under the same applied wheel load under the condition that the new slab rested directly on the subgrade without the old slab between it and the subgrade

f_2 = maximum tensile stress in the lower (old) slab resulting from the same wheel load applied to the upper (new) slab which rests on the lower (old) slab

f_3 = maximum tensile stress in the lower (old) slab resulting from the same wheel load being applied directly to it, without the interposition of the upper (new) slab between it and the load

h_1 = thickness of the upper (new) overlay slab

h_2 = thickness of the lower (old) slab

E_1 = modulus of elasticity of the upper (new) slab

E_2 = modulus of elasticity of the lower (old) slab

The stresses f_0 and f_3 would be computed by the Westergaard or Pickett procedure for a single slab. Palmer's paper suggests that this method is applicable where a separating course is placed between the two slabs provided the thickness of separation does not exceed 6 in. It would appear that the equations might be modified to fit the condition where no separating course is used and where some friction or bond would be present to increase the rigidity and structural capacity of the system.

So far as is known, there has been no experimental evaluation of Eq. (4) and (5) using strain gages or other methods. Such a study could well include an investigation of what modification, if any, is needed to make them applicable when no separation course is used. It is hoped that such a study soon can be undertaken.

Marcus and Palmer point out that Eq. (4) and (5) permit determination of stresses in both the upper and lower slabs, thus making it possible to avoid overstressing in either slab. It is also possible to select a different working stress for each slab if that seems appropriate.

JOINT LOCATION

In addition to slab thickness, there are a number of other questions to which the designer of concrete resurfacing needs answers. Perhaps the one of first importance is the matter of jointing. Observations of resurfacing pavements in service have led to the conclusion that with no separation course, or with a relatively thin one, most of the joints and cracks in the old pavement will be reflected or reproduced in the resurfacing. Accordingly, it is usually thought best to match all joints in the lower slab with joints in the resurfacing slab. These have not always been of the same type as in the old slab. For example, contraction joints have been used above old expansion joints, especially when the latter had become pretty well closed during the years of use. Similarly, tied (hinged) joints might well be used over an old untied longitudinal joint.

It is not necessary to place joints over all cracks in the old slab. Where there has been considerable cracking in the old slab, distributed steel has frequently been used in the overlay to prevent reflection cracks from becoming points of weakness. This measure seems to have been effective and is generally desirable. It has not prevented reflection cracks, but when held tightly closed they are considered objectionable mainly from the standpoint of appearance.

REINFORCEMENT OF OVERLAY

Reinforcement serves the same purposes in concrete overlays as it does in original construction. Its design should follow the "Recommended Practice for Design of Concrete Pavements (ACI 325-58)."* If distributed steel was used in the original pavement and no wide cracks have formed to indicate breakage of the steel, it has been common practice to base the design of the

*ACI JOURNAL, July 1958, *Proc.* V. 55, pp. 17-52, or see *ACI Standards, 1958.*

reinforcement in the resurfacing on the thickness of the resurfacing slab alone. If there was no distributed steel in the old slab, or if wide cracks indicate it to be broken, the steel in the resurfacing has been based on the combined thickness of the two slabs.

WIDENING COMBINED WITH RESURFACING

Old concrete pavements that require resurfacing often need widening also. While the number was not definitely established, a considerable proportion of the more than 300 projects covered in the survey mentioned earlier were combined resurfacing and widening projects. It appears that, far from causing troublesome complications, the construction of widening along with resurfacing offers certain definite advantages. Most important of all is the opportunity thus offered to construct the widening integral with the resurfacing, thickening the widening to that required for a single thickness slab to carry the expected load. This prevents the occurrence of a thin edge of the resurfacing which might constitute a point of weakness when curling action caused it to separate and become unsupported by the edge of the old slab.

Construction of a combined resurfacing and widening involves no special problems. In fact, the setting of forms is usually simplified and other operations are not made more difficult. This committee is concerned primarily with design, but the designer must give consideration to construction problems.

SEPARATION COURSE

The question of whether to use a separation course between the old and new slabs is one on which there is wide disagreement. Studies by different agencies have shown a definitely greater structural capacity when no separation course is used. Conversely, there seems little doubt but that the amount of reflection cracking can be reduced or even eliminated by use of separating courses of substantial thickness. Separating courses may be made necessary by changes in grade or by excessive irregularities in the old pavement which demand a leveling course before the new slab is constructed. Otherwise, a decision must be made as to how much added cost, due to the greater thickness of concrete as well as cost of the separation course itself, is justified to avoid the reflection cracks which are unsightly but which do little to limit the pavement's serviceability.

BONDED RESURFACING

The preceding discussion treats the normal concrete resurfacing in which no special efforts are made to effect bond to the existing pavement. Recently there has come into use a bonded type of resurfacing. Special steps are taken to prepare the old surface and to place the new in a manner which will effect a bond between the two. The question of structural design of bonded resurfacings has not arisen as they have been used, to date, only to correct an un-

satisfactory surface condition where no strengthening of the old slab was thought necessary.

At present, it would appear that an old slab with a bonded resurfacing can be treated as having a structural capacity equal in every way to that of a single slab with thickness equal to the total thickness of old slab plus the bonded resurfacing.

SUBGRADE CONDITIONS

A factor which must be considered in the design of concrete resurfacing is the condition of the underlying subgrade. It is obvious that where the subgrade has failed, the failure will not be corrected by placing an additional layer of concrete. In this case it will be better to break up the old slab, utilizing it for subgrade treatment, adding more subbase if needed, and construct a new pavement thereon.

CONCLUSION

Committee 325 has been assigned the task of developing a recommended practice for the design of structural details of concrete highway and airport pavement. Having presented to the Institute a recommended practice for normal single course construction, the committee has been reorganized to study special phases of concrete pavement design. Subcommittee VIII was created to study the design of concrete resurfacing. The foregoing is not intended to represent the recommendation of the subcommittee but is a statement of the situation at the beginning of the study. Comments and information are invited from engineers with applicable experience and data. As rapidly as practical, the subcommittee will study the available facts and prepare a recommended practice for the structural design of concrete resurfacing.

With the present knowledge of the subject it is anticipated that the initial recommendation will be subject to future revision. It is believed a report should not be delayed until the completion of the research needed for a final report. Rather, it is planned to present a report based on the best information available. Performance of resurfacings in service gives assurance that recommendations can be made which will insure good results. When still better information has been developed it will be possible to revise the committee's first recommendation.

Discussion of this report should reach ACI headquarters in triplicate by December 1, 1958, for publication in the March 1959 JOURNAL.

Title No. 55-20

Final report of a project sponsored by the Reinforced Concrete Research Council at the University of California

Strength of Concrete Under Combined Stresses*

By B. BRESLER† and K. S. PISTER‡

SYNOPSIS

A criterion for failure of plain concrete subjected to combined stresses was established from tests of 65 tubular specimens tested to failure under various combinations of shearing and compressive stress. A procedure for determining the shearing strength of a special class of rectangular reinforced concrete beams without web reinforcement was developed. Excellent correlation was obtained between calculated and observed shearing strength of a limited group of beams.

INTRODUCTION

Failure of structural elements usually occurs under complex loading conditions; hence the state of stress at failure is generally complex. For this reason, knowledge of criteria of failure for materials subjected to combined stresses is important. Reinforced concrete elements, such as beams, rigid frames, plate floor slabs, and shell roofs, and similar prestressed elements are subjected to loads producing combinations of normal and shearing stresses. Therefore, investigation of strength limitations for concrete subjected to combined stresses becomes particularly important if the ultimate strength theories are to be accepted.

Investigation of failure in plain concrete is complicated by the nonhomogeneity and nonisotropy of the material, and in reinforced concrete it is further complicated by the presence of reinforcement. It has been often stated that presence of reinforcement in concrete alters the nature of the material and, therefore, changes the criteria for failure. Indeed, in considering the mechanism of failure, that is, the progress from initial local failure to ultimate collapse, both the bond between concrete and reinforcement and the deformation or failure in the reinforcement play an important role. It seems reasonable, however, that the conditions producing local failure in concrete are essentially the same for both plain and reinforced concrete, and therefore knowledge of failure criteria for plain concrete would be valuable for predicting collapse strength of reinforced or prestressed concrete structural elements of any size or shape.

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The study reported here had two principal objectives. The first was to verify the validity of a failure criterion for plain concrete based on a relationship between normal and shearing mean stresses. Such a hypothesis had been formulated by the writers as a result of a preliminary investigation[1] but in view of the limited data obtained at that time, further verification was deemed desirable.

The second objective was to apply the failure criterion to the determination of shearing strength of structural elements. In this study an excellent correlation between the failure criterion and the shearing strength of a special class of simply supported rectangular reinforced concrete beams without web reinforcement was obtained.

TEST PROGRAM

Scope

The experimental investigation was limited to the study of strength of plain concrete subjected to various combinations of compression and shear. All test specimens were of the same tubular shape: 9-in. outside diameter, 6-in. inside diameter, and 30 in. long. Stresses at failure were determined for specimens subjected to pure axial compression, pure torsion, and torsion combined with compression of 20, 40, 60, and 80 percent of pure compressive strength. Three concrete mixes were used in this investigation with nominal compressive strengths of 3000, 4500, and 6000 psi at the age of 28 days. All specimens were tested at approximately 28 days after casting.

The test program was divided into three series. The objective of tests in Series I was to determine the effect of end conditions imposed by a special torque band on the pure compressive strength of specimens. For each mix one specimen was tested without the torque band and one with the torque band clamped on the ends. In all subsequent tests, Series II and Series III, all pure compression test specimens had torque bands clamped on the ends.

Series II included tests of duplicate specimens for each of the loading conditions in the program. At the conclusion of the tests in this series it became evident that due to normal variations in the performance of plain concrete and due to initial difficulties in the test procedure, some of the tests of Series II should be repeated to verify the results. In Series I and II "sequence" loading was used, in which a predetermined amount of compression was applied to the specimen before producing failure in torsion. It seemed desirable to investigate the effect of "proportional" loading in which compression and torsion were increased in proportional steps. Therefore, Series III included a number of tests, some of which were designed to verify the results of Series II, and some to determine the effect of proportional loading on the strength of concrete.

In all, 65 specimens were tested, six in Series I, 36 in Series II, and 23 in Series III. The specimens were designated by a code intended to identify the series number, the nominal strength of the mix, and the loading conditions. Thus, II 4.5 TC.4A refers to a specimen in Series II, made of 4500-psi

concrete, subjected to torsion "T," combined with compression "C" of 0.4 of pure compressive strength; "A" designates the first specimen of two made from a given batch. Specimens subjected to sequence loading were marked "A" and "B"—first or second, and specimens subjected to proportional loading were marked "P."

Materials and mix proportions

Type I portland cement was used in this investigation. The cement was purchased in a single lot, blended for uniformity and stored in steel drums. A blend of three sands was used as fine aggregate, the blend having been chosen for its soundness, uniformity, and gradation. The proportion of each sand was weighed out separately for each batch to insure better uniformity of gradation. River gravel was used as coarse aggregate, varying in size from sieve No. 4 to ½ in. Petrographic descriptions of the mineral content of the sands and the coarse gravel are shown in Table 1, and the results of sieve analyses are shown in Table 2. Mix proportions were selected to give the desired strength of concrete, and several trial batch mixes were made before the proportions were finally selected. In preparing the specimens for Series I, it was found that mixes with nominal strengths of 4500 and 6000 psi seemed too wet, and the mixes were reproportioned slightly so that specimens in Series II and III with

TABLE 1—PETROGRAPHIC DESCRIPTION OF SAND AND GRAVEL

Mineral	Mineral composition of sands, percent			Mineral	Mineral composition of Fair Oaks gravel, percent
	Antioch	Monterey No. 8	Monterey No. 20		
Quarts	79.5	87.0	81.4	Basalt, andesite, dacite	43
Green horneblende	7.9		5.9	Quarts	35
Feldspar	5.4	10.8	4.9	Schist (amphibolite and quarts mica)	9
Magnetite	2.6	1.6	1.9	Horneblende	9
Miscellaneous	4.6	0.6	5.9	Chert	4

TABLE 2—SIEVE ANALYSES OF SANDS AND GRAVEL

Sieve size	Sand, percent retained				Gravel, percent retained
	Antioch	Monterey No. 8	Monterey No. 20	Blend*	
½ in.					0
⅜ in.					24.4
No. 4					99.4
8			2.2	1.0	100
16			87.5	41.2	
30	12.5	6.3	99.5	51.1	
50	35.9	75.3	99.9	81.0	
100	69.4	99.7	100.0	95.3	
Fineness modulus	1.18	1.81	3.89	2.70	6.24

*Blend: 15 percent Antioch; 38 percent Monterey No. 20; 47 percent Monterery No. 8.

TABLE 3—MIX PROPORTIONS

Property or proportion	Series I, II, III	Series I	Series II, III	Series I	Series II, III
Nominal compressive strength, psi	3000	4500	4500	6000	6000
Water-cement ratio, by weight	0.69	0.51	0.52	0.42	0.41
Water-cement ratio, gal. per sack	7.8	5.8	5.9	4.7	4.6
Sand-cement ratio, by weight	4.03	2.84	2.92	1.98	1.98
Gravel-cement ratio, by weight	4.36	3.46	3.57	2.72	2.73
Cement content, sacks per cu yd	4.26	5.58	5.58	7.22	7.22
Slump, in., average	1.7	—	2.3	—	2.5
Slump, in., range	1.2–2.5		1.6–2.9		1.6–3.0

nominal strengths of 4500 and 6000 psi were made with a mix differing slightly from those in Series I. Mix proportions, shown in Table 3, are based on corrected quantities for saturated surface dry condition of all aggregates.

Fabrication and curing of specimens

Two-cu ft batches were mixed in a horizontal drum Lancaster mixer. All batching was done by weight, the materials being used in air dry condition. The dry materials were first blended in a mixer 1 min, then water was added and the batch mixed for 3 min. The slump was measured about 7 min after the start of the mixing operation. Average values and the range of variation of slump for each of the mixes are shown in Table 3.

All tubular specimens were cast in a vertical position in a split metal mold with a wooden core consisting of four wedged pieces held in a rubber tube. The mold was mounted on a vibrating table and the concrete was placed in the mold continuously through a funnel. During placement the concrete was continuously rodded and vibrated, and the placement was completed in 4 or 5 min. After several trials, this casting procedure was selected as one which resulted in the most uniform concrete structure without noticeable segregation or excessive voids.

Loading apparatus

The loading apparatus for testing the tubular specimens consisted of a 400,000-lb Baldwin Southwark universal testing machine to apply axial compression, and a special frame assembly to apply torsional loads. The torsion-loading frame assembly (Fig. 1 and 2) consisted of two end beams held together by four ¾-in. diameter tie rods, two cross beams which transmitted the loads to torque bands clamped on the ends of the specimens, a hydraulic jack to

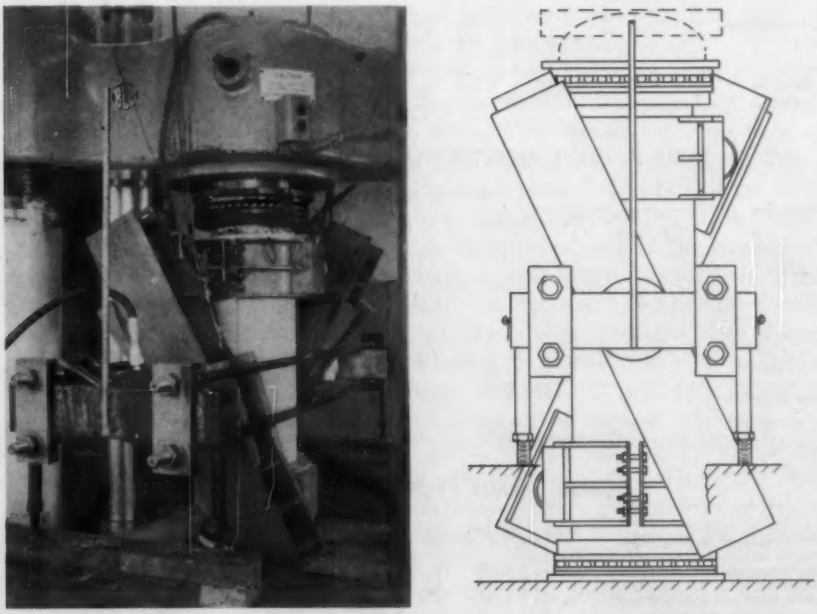

Fig. 1—Specimen set up in testing machine, with end view of loading frame assembly detailed at right

END

apply the load to one of the cross beams, and a proving ring to measure the applied load. The diagonally opposite cross beams were loaded in such a manner that the end reactions formed opposite couples at each end of the specimen. These couples were transmitted through the torque bands to the specimen.

To eliminate frictional restraint to rotation at the end of the specimen special thrust-bearing assemblies were used. Each thrust-bearing assembly consisted of a Rollway Precision T-46 thrust bearing mounted on a base with a short centering shaft, and modified by replacing the rollers with 96 chrome alloy steel ¾-in. diameter bearing balls. The amount of frictional resistance to rotation offered by the thrust bearings was measured as a function of the thrust in several calibration tests and the torque required to produce first slip in the bearings was

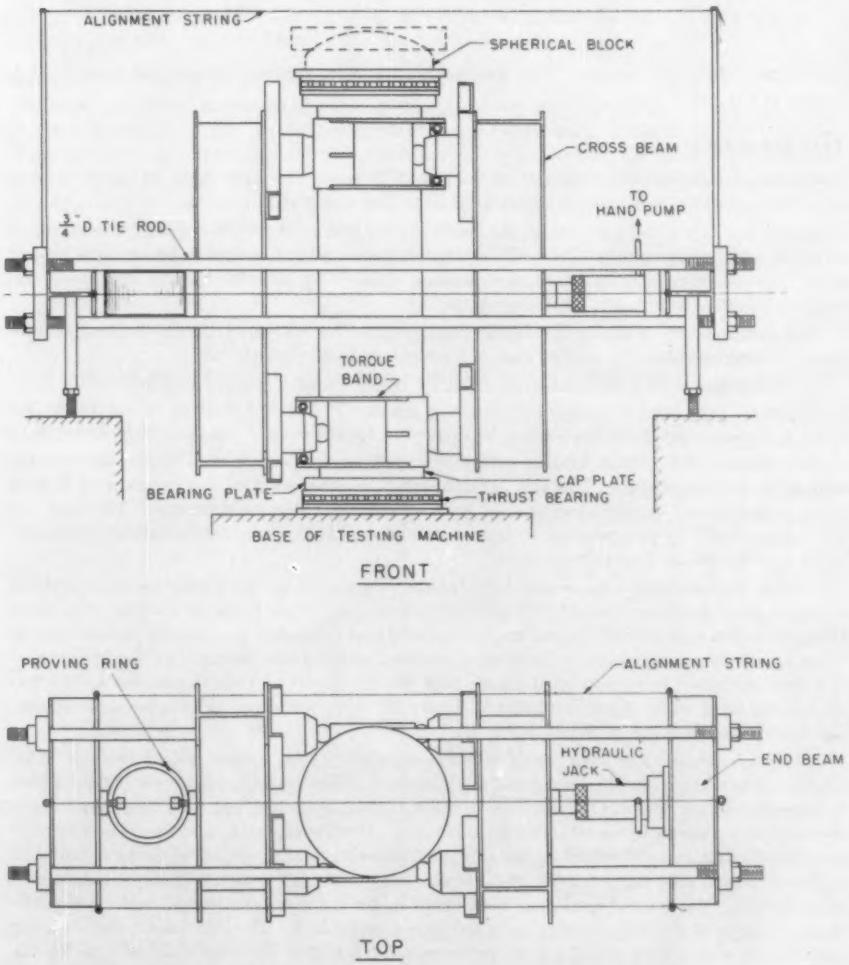

Fig. 2—Top and front views of loading frame assembly

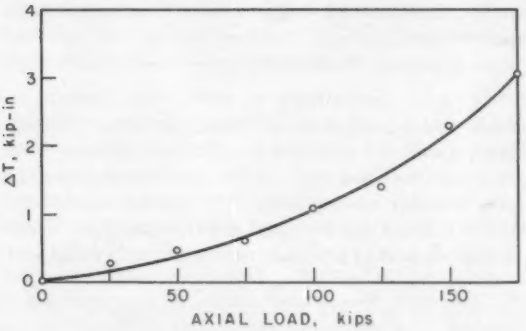

Fig. 3—Thrust bearing friction at first slip

called the "slipping" torque. The relationship between slipping torque and axial load is shown in Fig. 3.

Test procedure

Specimens were scheduled for test at the age of 28 days. In some cases due to unforeseen experimental difficulties testing was delayed from 1 to 4 days.*

On the day scheduled for testing, the specimens were removed from the fog room and kept damp by wet burlap covers. The ends of tubular specimens and control cylinders were capped with hydrostone using 1 in. thick steel bearing plates. In all cases tubular specimens and control cylinders were tested at the same age.

Pure compression—Tubular specimens were tested in a 400-kip Baldwin Southwark universal testing machine at a loading rate of approximately 15 kips per min.

Torsion bands were positioned 17½ in. apart by means of a spacer, and four bolts, ½ in. in diameter, were used to clamp the bands in place. To prevent slipping of the bands the bolts were tightened until the tension in them was approximately equal to yield strength.

Pure torsion—The torsion loading assembly was supported on two steel tables, one on each side of the testing machine. The jack, proving ring, cross beams, and end beams were aligned using a plumb bob, combination square, level, and special alignment strings. The load was applied gradually by pumping the hydraulic jack manually, the total time of loading to failure being approximately 5 min.

Compression combined with torsion—Specimens were placed in the testing machine and the torsional loading frame was aligned as described above. Two types of loading were used. Using sequence type of loading, the specimens were first subjected to a predetermined amount of an axial compressive load—a portion of nominal compressive strength of the specimen—and then subjected to torsion until failure took place. The total time to produce failure was approximately 5 min. Four specimens in Series III were subjected to proportional loading. The load increments are shown in Table 4.

Control specimens—For each batch of concrete, six 3 x 6-in. cylinders and two 6 x 12-in. cylinders were prepared. The compressive strength of all the control cylinders was determined in accordance with ASTM C 39-44. One of the 6 x 12-in. cylinders was used to determine the stress-strain relationship up to about 80 percent of ultimate strength, and then the specimen was loaded to failure. To correlate the compressive strength of the mixes with the modulus of rupture for each mix, three 6 x 6 x 20-in. beams were made and tested at 28 days under third-point loading. All control specimens were cured under the same conditions as the test specimens. Results of control specimen tests are shown in Table 5. The relationship between the compressive strength of the 3 x 6-in. cylinders and 6 x 12-in. cylinders appears to be in-

*Thirty-nine specimens were actually tested at 28 days, seven were tested at 29 days, 12 at 30 days, one at 31 days, and six at 32 days.

TABLE 4—PROPORTIONAL LOADING

Load increments, percent of load at failure, for specimen indicated

III 3TC.4P		III 3TC.8P		III 4.5TC.4P		III 4.5TC.8P	
Compression	Torsion	Compression	Torsion	Compression	Torsion	Compression	Torsion
18	11	18	12	21	14	21	12
37	22	37	25	41	28	41	24
56	33	56	38	62	42	62	36
74	55	74	63	83	70	83	59
100	100	100	100	100	100	100	100

fluenced by richness of mix and water-cement ratio as shown in Fig. 4. The ratios of 3 x 6-in. to that of 6 x 12-in. cylinder strengths are 1.21, 1.11, and 1.06 for water-cement ratios of 0.41, 0.52, and 0.69 respectively. In view of this variation, test data obtained in this study are correlated with both the 3 x 6-in. and the 6 x 12-in. cylinder strength.

Measurements and stress calculations

Outside and inside diameters of each tubular specimen were determined at several points at each end and the average value was calculated for the specimen. The total torque T applied to the specimen at failure was determined as the couple formed by the cross-beam reactions to the jack load. The torque ΔT resisted by friction at each end was assumed equal to the slipping torque at a given compression load P, and was determined by calibration. The net torque on the specimen was defined as $T_n = T - \Delta T$.

Compressive and shearing stresses were calculated from conventional equations:

$$\left.\begin{aligned} \sigma &= P/A \\ \text{and} \qquad \tau &= T_n r_o/J \end{aligned}\right\} \qquad \text{.....(1)}$$

where σ = average compressive stress, and τ = maximum shearing stress at the outer sur-

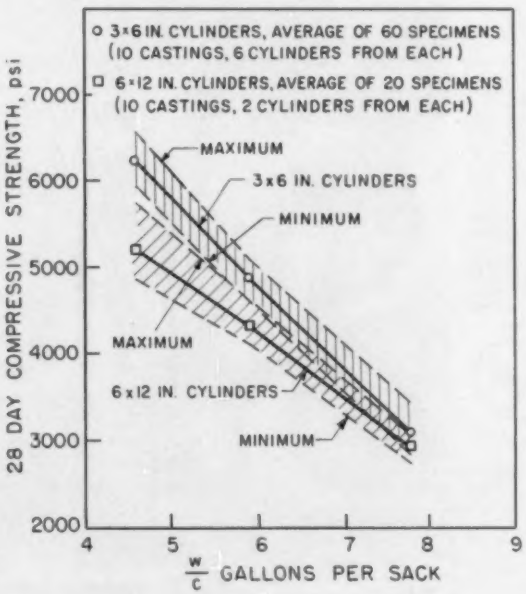

Fig. 4—Compressive strength of 3 x 6-in. and 6 x 12-in. concrete control cylinders

face of the cylinder. Strain measurements made in a previous study indicated that linear shearing stress distribution is valid for pure torsion specimens. It is assumed that linear shearing stress distribution is also valid for the combined loading conditions. Also, the ratio of average shearing stress to maximum shearing stress is 0.83:1.00, indicating reasonably small variation in stress distribution across the thickness. Therefore, the maximum nominal shearing stress τ was used as an indication of shearing stress in concrete. Calculated values of compressive and shearing stresses at failure are shown in Table 6.

Modes of failure

After completion of each test the specimen was photographed, taking two or more views of the failure, so that a complete record of the types of failure was obtained. The five principal modes of failure are illustrated in Fig. 5.

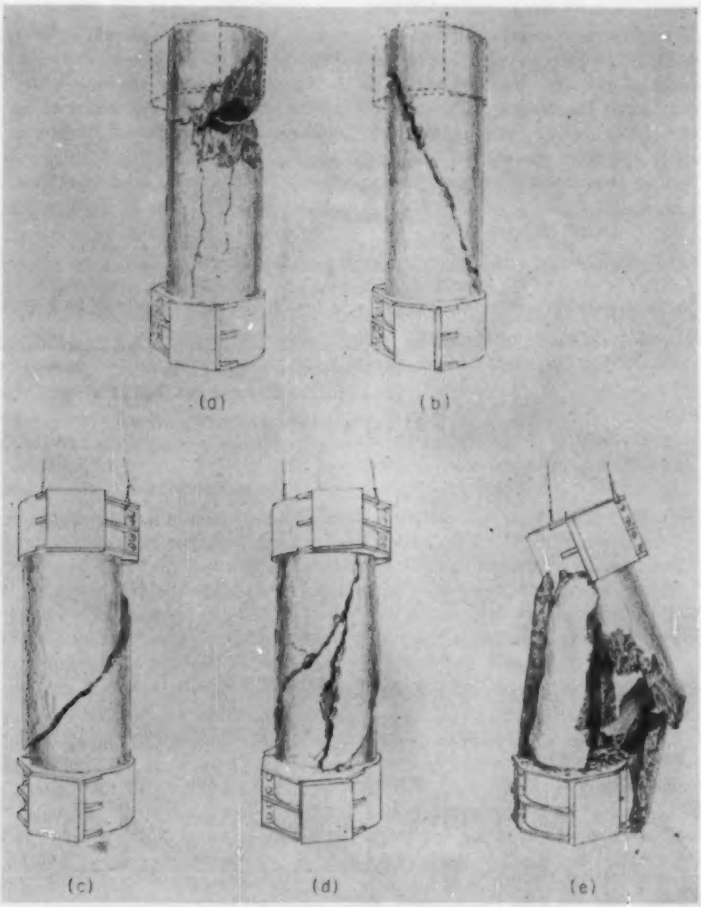

Fig. 5—Modes of failure

The specimens loaded in pure compression exhibited two modes of failure: one was splitting along vertical planes, indicated by vertical cracks in Fig. 5a, and the other was failure along somewhat inclined planes as shown in Fig. 5b. No consistent correlation between type of failure and nominal compressive strength of concrete could be noted. In some cases, due to the presence of torque bands, a secondary failure was observed, such as transverse circumferential splitting, accompanied by crushing of concrete and spalling along a somewhat inclined plane within the thickness of the tube (Fig. 5a).

The specimens loaded in pure torsion failed by splitting along the helicoidal surface making an angle somewhat less than 45 deg with the longitudinal axis (Fig. 5c).

Two types of failure occurred in specimens subjected to combined compression and torsion. One, shown in Fig. 5d, consisted of two principal cracks; a steep one inclined approximately 15 deg to the longitudinal axis, and another making an angle of approximately 45 deg with that axis. The inclinations of the cracks varied with the relative magnitudes of the compressive load, but no consistent relationship between angle of inclination of the crack and the loading condition or the nominal compressive strength of concrete could be noted. Furthermore, it was not apparent which of the two cracks formed first and which was secondary. Another prevalent type of failure was that of complete and sudden breakdown along several planes,

TABLE 5—AVERAGE STRENGTH OF CONTROL SPECIMENS

Compressive strength

Specimen No.	Compressive strength, psi		Specimen No.	Compressive strength, psi		Specimen No.	Compressive strength, psi	
	3 x 6-in. cylinder	6 x 12-in. cylinder		3 x 6-in. cylinder	6 x 12-in. cylinder		3 x 6-in. cylinder	6 x 12-in. cylinder
I 3C	3140	2910	I 4.5C	4850	4070	I 6C	6520	5280
II 3C	2890	3020	II 4.5C	5080	4610	II 6C	6560	5770
—	—	—	—	—	—	III 6C	6130	5300
II 3T	3270	3070	II 4.5T	5030	4350	II 6T	6500	5380
III 3T	3440	2960	III 4.5T	4910	4510	III 6T	6310	5060
II 3TC.2	3130	2990	II 4.5TC.2	4850	4250	II 6TC.2	6350	5020
III 3TC.2	2830	2820	—	—	—	—	—	—
II 3TC.4	3130	2840	II 4.5TC.4	4230	4310	II 6TC.4	6210	4960
III 3TC.4	3220	3060	III 4.5TC.4P	4620	4360	III 6TC.4	5940	5230
—	—	—	III 4.5TC.4A	4900	4330	—	—	—
II 3TC.6	2940	2740	II 4.5TC.6	4580	4100	II 6TC.6	5980	5040
III 3TC.6	3130	2950	III 4.5TC.6	4990	4450	III 6TC.6	6010	4840
II 3TC.8	3070	2900	II 4.5TC.8	4980	4150	II 6TC.8	6340	5420
III 3TC.8	3220	3060	III 4.5TC.8A	4990	4450	III 6TC.8	6010	4840
—	—	—	III 4.5TC.8P	4620	4360	—	—	—
AVERAGE	3120	2940		4820	4330		6240	5180

Flexural strength

3000-psi mix		4500-psi mix		6000-psi mix	
Specimen No.	Modulus of rupture, psi	Specimen No.	Modulus of rupture, psi	Specimen No.	Modulus of rupture, psi
B 3-A	438	B 4.5-A	656	B 6-A	792
B 3-B	432	B 4.5-B	702	B 6-B	748
B 3-C	431	B 4.5-C	619	B 6-C	743
AVERAGE	434		660		761

TABLE 6—CONVENTIONAL COMPRESSIVE AND SHEARING STRESSES AT FAILURE*

Nominal strength, 3000 psi			Nominal strength, 4500 psi			Nominal strength, 6000 psi		
Specimen	σ	τ	Specimen	σ	τ	Specimen	σ	τ
I 3C A	2950	—	I 4.5C A	4240	—	I 6C A	5170	—
B	2790	—	B	4450	—	B	4430	—
II 3C A	3030	—	II 4.5C A	4080	—	II 6C A	5570	—
B	3000	—	B	4320	—	B	6210	—
—			—			III 6C A	5980	—
—			—			B	5620	—
II 3T A	—	200	II 4.5T A	—	210	II 6T A	—	340
B	—	200	B	—	280	B	—	270
III 3T A	—	270	III 3T A	—	400	III 6T A	—	460
B	—	240	B	—	340	B	—	460
II 3TC.2 A	610	440	II 4.5TC.2A	830	430	II 6TC.2 A	1180	850
B	610	430	B	830	490	B	1180	790
III 3TC.2 A	610	440	—			—		
B	610	560	—			—		
II 3TC.4 A	1220	660	II 4.5TC.4A			II 6TC.4 A	1660	860
B	1220	660	B	1660	920	B	1660	870
III 3TC.4 P	1220	660	III 4.5TC.4P	1670	830	III 6TC.4 A	2350	1080
			A	1660	990	B	2350	1200
II 3TC.6 A	1830	570	II 4.5TC.6A	2490	920	II 6TC.6 A	3500	880
B	1840	740	B	2510	1150	B	3520	1080
III 3TC.6 A	1830	680	III 4.5TC.6A	2490	1070	III 6TC.6 A	3540	1170
B	1840	830						
II 3TC.8 A	2460	660	II 4.5TC.8A	3370	800	II 6TC.8 A	4740	800
B	2440	720	B	3340	1020	B	4700	1000
III 3TC.8 P	2440	790	III 4.5TC.8A	3350	1130	III 6TC.8 B	4710	1070
			P	3320	490			

* σ = compressive stress, psi; τ = shearing stress, psi.

as shown in Fig. 5e. The specimen disintegrated into numerous pieces. This type of failure was particularly prevalent for the 6000-psi concrete under practically all combinations of compression and torsion. It was not limited to the 6000-psi concrete, however, as specimens of other concretes subjected to high proportions of compressive failure load also exhibited a similar type of failure.

DETERMINATION OF A FAILURE CRITERION FOR PLAIN CONCRETE

Introduction

In formulating a law of strength under combined states of stress, agreement must be reached as to what constitutes failure. Criteria such as yielding, initiation of cracking, load carrying capacity, and extent of deformation have been used to define failure. The type of specimen and type of test have an appreciable influence on the resulting type of failure and therefore correlation of tests in which the various definitions of failure have been employed is difficult. Furthermore, the application of criteria based on test specimens to full size structural elements is made difficult by the necessity of a definition of failure for both test specimen and full size element, which definition need not be the same for each. In this investigation failure is defined as the ultimate load carrying capacity of the test specimen. This seems reasonable

in view of the statically determinate nature of the stress distribution up to and including failure. Determination of microcrack formation, yielding, or measurement of limiting strain (difficulty intensified by creep and plastic flow) all seem to be less desirable indicators of failure for concrete specimens used in this study. Ideally, in any program designed to establish the effect of state of stress on strength of concrete, the law defining failure should be capable of predicting strengths of different types and sizes of test specimens. This problem involves the nature of loading, boundary conditions at loaded surfaces, stress distribution at critical sections, stress or strain rate, and stress history of specimen including effects of curing and shrinkage. Although these factors per se may not affect the actual application of the failure law to structural elements, they are determinative in explaining and correlating test data taken from various sources.

The nature and structure of the material must be identified. In concrete the physical and mechanical properties of the aggregate and of the cement paste, and the nature of deformations and of volume changes due to temperature or shrinkage are significant factors affecting the behavior of the material. Obviously the mix proportions must also be included.

Notation

The letter symbols used in this report are generally defined when they are introduced. The most frequently used symbols are listed below for convenient reference.

A = cross-sectional area, or numerical coefficient
A_c = effective area resisting shear
A_s = area of longitudinal reinforcement
a = shear span; normally distance between support and applied concentrated load
b, d = width and effective depth of rectangular beam, respectively
C = compression force due to internal moment couple
f_c' = compressive strength of 6 x 12-in. concrete test cylinders
f_c'' = compressive strength of 3 x 6-in. concrete test cylinders
f_y = yield strength of longitudinal reinforcement
I_1, I_2, I_3 = principal stress invariants
j = ratio of internal moment arm to effective depth d
J = polar moment of inertia
k = ratio of depth of compression zone to effective depth d
M = bending moment
p = (A_s/bd)
P = axial load
T = tension force due to internal moment couple
V_t = measured shearing force at failure, based on test results
V_u = calculated shear force at failure, based on shear criterion
V_f = calculated shear force at failure, based on flexure criterion
σ = normal stress
τ = shearing stress

Review of failure theories

In addition to the review of theories of failure applicable to concrete found in Reference 1, the following experimental investigations are cited.

A series of biaxial compression tests of 15-cm concrete cubes conducted by Wastlund[2] indicates that the intermediate principal stress is determinative

in the failure of the specimen, thus contradicting the Mohr theory for biaxial states of stress in the compression-compression quadrant.

McHenry and Karni[3] conducted tests on hollow plain concrete cylinders subjected to hydrostatic pressure and axial compression. The results of this study are generally in agreement with the failure law proposed by the authors.[1]

Invariant formulation of failure laws

A failure criterion based upon state of stress must be an invariant function of the state of stress, that is, independent of the choice of the coordinate system by which stress is defined. One method of representing such a function is to utilize the principal stresses. Thus, $F(\sigma_1, \sigma_2, \sigma_3,) = 0$ is frequently used to indicate the general functional form of failure law, the actual determination of the function being left to theoretical conjecture or experimentation. However, unless some restriction is placed on the function, considerable difficulty is experienced in accomplishing such an experimental determination. In the past, to give a few examples, the function has been associated with the normal and shear stresses on the plane of failure neglecting the influence of the intermediate principal stress, or with effective stress, or with assumed expressions based upon experimental data.

In the general case of a polydimensional state of stress this method of establishing a failure law is particularly difficult to pursue since three independent parameters, the principal stresses, are involved. Furthermore, it is difficult to supply a physical explanation of failure on this basis.

An alternative way of describing a failure law has proved useful. It is known that any invariant symmetric function of the state of stress (e.g., a criterion of failure) can be expressed in terms of the three principal stress invariants. Thus, one can write

$$F(I_1, I_2, I_3) = 0 \qquad (2)$$

where

$$\left.\begin{aligned} I_1 &= \sigma_1 + \sigma_2 + \sigma_3 \\ I_2 &= \sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1 \\ I_3 &= \sigma_1\sigma_2\sigma_3 \end{aligned}\right\} \qquad (3)$$

In formulating criteria for yielding of ductile materials subjected to combined states of stress it has been found that other invariants of the state of stress, which of course can be formed by proper combinations of the principal invariants, are more susceptible of physical interpretation. An explanation of such invariants which is independent of the properties of a specific material or class of materials is contained in a paper by Novozhilov.[4] It seems reasonable, therefore, to utilize these invariants in formulating a law of failure of a brittle material such as concrete.

Mean shearing stress and mean normal stress

The concept of mean stress seems particularly attractive because it provides a physical interpretation of invariants used. Consider an infinitesimal spherical element of volume. At any point on the surface of this element the state of stress can be expressed in terms of a shearing stress τ_s and normal stress σ_s. The mean value of the shearing stress can be based upon stresses existing on all possible planes of orientation through the point by carrying out the averaging process over the spherical surface. Although shearing stress can be either positive or negative, the sign has no significance with respect to physical mechanism of failure, and it is expedient to take the average in the sense of the root mean. Thus,

$$\tau_a = \operatorname*{Lim}_{S \to 0} \left[\frac{1}{S} \int_S \tau_s^2 dS \right]^{1/2} \quad \ldots\ldots (4)$$

where S denotes the surface of the spherical element of volume. Carrying out the indicated operations leads to Eq.(5).

$$\tau_a = \frac{1}{\sqrt{15}} \left[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \right]^{1/2} \quad \ldots\ldots (5)$$

or in terms of the principal invariants,

$$\tau_a = \sqrt{\frac{2}{15}} \left[I_1^2 - 3I_2 \right]^{1/2} \quad \ldots\ldots (6)$$

In the case of the normal stress σ_s on the surface of the spherical element the sign of the stress is significant, i.e., whether the stress is tensile or compressive certainly influences the mechanism of failure. Accordingly the mean stress can be defined by Eq.(7).

$$\sigma_a = \operatorname*{Lim}_{S \to 0} \left[\frac{1}{S} \int_S \sigma_s \, dS \right] \quad \ldots\ldots (7)$$

Evaluation of this expression gives

$$\sigma_a = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) = \frac{1}{3} I_1 \quad \ldots\ldots (8)$$

The third "plasticity" invariant has been shown by Novozhilov to represent the ratio of the mean shearing stress to the maximum shearing stress, which varies over very narrow limits. Therefore, its effect on the formulation of a

failure law would be of secondary importance. Consequently, it is possible to write a failure law in the form

$$\left.\begin{array}{l} F(\sigma_a, \tau_a) = 0 \\ \text{or} \quad \tau_a = f(\sigma_a) \end{array}\right\} \qquad (9)$$

For an isotropic, elastic, ductile material following Hooke's law it is well known that experimental evidence discloses an independence of σ_a and τ_a for the condition of yielding: futhermore, τ_a is directly proportional to the energy of distortion.[5] However, for materials such as concrete, it does not appear to be expedient to interpret τ_a in terms of elastic energy. It is more suitable to seek a physical interpretation of the parameters σ_a and τ_a that is independent of the properties of a specific material or class of materials.

Finally, the failure criterion in the form $\tau_a = f(\sigma_a)$ accounts for the effect of the intermediate principal stress σ_2, and thus is a natural generalization of the Mohr theory of failure which neglects its effect, and whose importance for concrete has been shown in the experiments of Wastlund, among others.

The form of the function $f(\sigma_a)$ is left to experiment. In this investigation two trial functions have been utilized, a linear equation and a quadratic equation. Which form is preferable is a question that additional substantiating tests and applications of the theory to structural elements will ultimately have to answer.

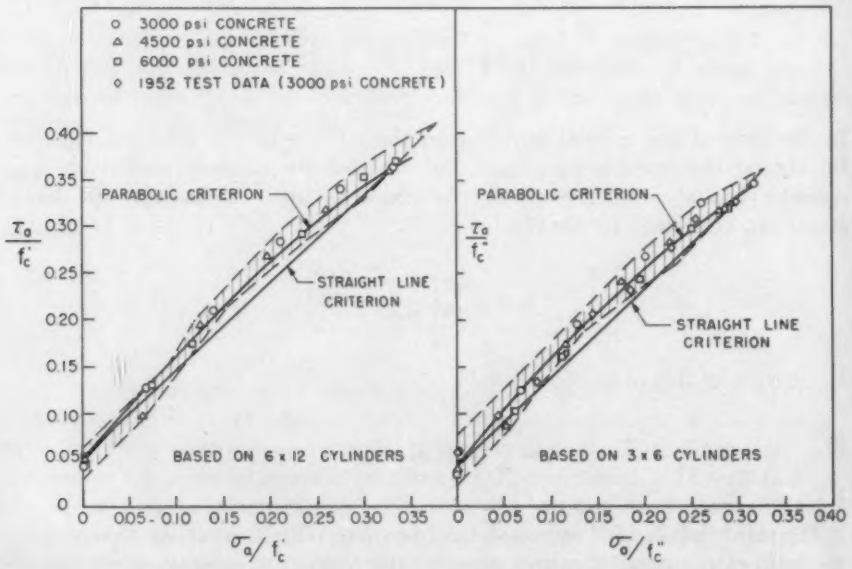

Fig. 6—Relationship between normal and shearing mean stresses at failure

Test results

As noted elsewhere most of the specimens tested in this program were subjected to sequence loading. As seen from Table 6 the specimens tested under conditions of proportional loading (designated by "P") did not show any significant variation from those tested under the sequence loading. Therefore, no distinction is made between the two types of loading. Mean stresses defined by Eq.(5) and (8) were calculated and are listed in Table 7.

TABLE 7—MEAN COMPRESSIVE AND SHEARING STRESSES AT FAILURE*

Nominal strength, 3000 psi			Nominal strength, 4500 psi			Nominal strength, 6000 psi		
Specimen	σ_a	τ_a	Specimen	σ_a	τ_a	Specimen	σ_a	τ_a
I 3C A	980	1040	I 4.5C A	1410	1560	I 6C A	1720	1900
B	930	1030	B	1480	1640	B	1480	1630
II 3C A	1010	1120	II 4.5C A	1360	1500	II 6C A	1860	2050
B	1000	1100	B	1440	1590	B	2070	2290
						III 6C A	1990	2200
						B	1870	2070
II 3T A	—	130	II 4.5T A	—	140	II 6T A	—	220
B	—	130	B	—	170	B	—	170
III 3T A	—	170	III 3T A	—	260	III 6T A	—	290
B	—	150	B	—	220	B	—	300
II 3TC.2 A	200	360	II 4.5TC.2A	280	410	II 6TC.2 A	390	670
B	200	360	B	280	410	B	390	670
III 3TC.2 A	200	360	—			—		
B	200	420	—			—		
II 3TC.4 A	400	620	II 4.5TC.4A			II 6TC.4 A	550	830
B	400	620	B	550	850	B	550	830
III 3TC.4 P	400	620	III 4.5TC.4P	560	820	III 6TC.4 A	780	1100
			A	550	880	B	780	1160
II 3TC.6 A	610	760	II 4.5TC.6A	830	1090	II 6TC.6 A	1170	1410
B	610	830	B	840	1170	B	1170	1470
III 3TC.6 A	610	800	III 4.5TC.6A	830	1140	III 6TC.6 A	1180	1490
B	610	860	—			—		
II 3TC.8 A	820	1000	II 4.5TC.8A	1120	1340	II 6TC.8 A	1580	1820
B	810	1010	B	1110	1390	B	1570	1860
III 3TC.8 P	810	1030	III 4.5TC.8A	1120	1420	III 6TC.8 B	1570	1860
			P	1110	1360			

* σ_a = mean compressive stress, psi; τ_a = mean shearing stress, psi.

The experimental data indicating the relationship between mean normal and shearing stress-ratios is shown in Fig. 6. Points represent average values for a group of similar specimens and the shaded area represents the scatter of the data. The experimental data can be closely approximated by a quadratic parabola of the form:

$$\frac{\tau_a}{f_c} = A + B\frac{\sigma_a}{f_c} + C\left(\frac{\sigma_a}{f_c}\right)^2 \qquad (10)$$

where σ_a and f_c are taken positive when compressive. The lower bound of the test data is approximated by a straight line, Fig. 6. Numerical values

of coefficients A, B, and C in Eq.(10) determined empirically from data shown in Fig. 6 are given below.

Criterion	Numerical coefficients		
	A	B	C
Based on 6 x 12-in. cylinders, $f_c = f_c'$			
Parabola	0.050	1.224	0.826
Straight line	0.050	0.949	0
Based on 3 x 6-in. cylinders, $f_c = f_c''$			
Parabola	0.045	1.274	1.160
Straight line	0.045	0.941	0

For the special case of shearing stress τ combined with uniaxial compression σ the failure criteria defined by Eq.(10) can be transformed into equations relating conventional shearing and normal stress-ratios by substituting the following

$$\left.\begin{aligned} \sigma_a &= \frac{\sigma}{3} \\ \text{and} \quad \tau_a &= 0.366\,(\sigma^2 + 3\,\tau^2)^{1/2} \end{aligned}\right\} \qquad \text{(11)}$$

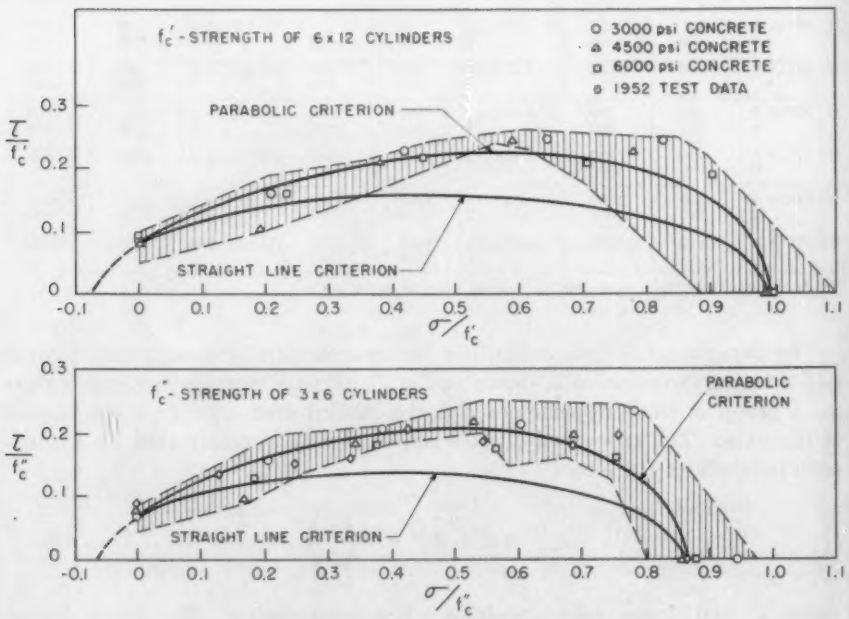

Fig. 7—Shear-compression strength

The transformation leads to an equation of the form:

$$\frac{\tau}{f_c} = 0.1\left[A' + B'\left(\frac{\sigma}{f_c}\right) + C'\left(\frac{\sigma}{f_c}\right)^2 + D'\left(\frac{\sigma}{f_c}\right)^3 + E'\left(\frac{\sigma}{f_c}\right)^4\right]^{1/2} \quad \ldots\ldots(12)$$

where the numerical coefficients for the failure criteria in terms of conventional stresses are given below.

Criterion	Numerical coefficients				
	A'	B'	C'	D'	E'
Based on 6 x 12-in. cylinders, $f_c = f_c'$					
Parabolic	0.62	10.10	5.80	−18.60	+2.09
Straight line	0.62	7.86	−8.46	0	0
Based on 3 x 6-in. cylinders, $f_c = f_c''$					
Parabolic	0.50	9.50	8.63	−27.20	4.13
Straight line	0.50	7.01	−8.85	0	0

The test data in terms of conventional stresses σ and τ and the empirical curves based on Eq.(12) are shown in Fig. 7.

Discussion

Taking into account the scatter of the data it is desirable to select a conservative failure criterion such as the "straight line" criterion, which can be expressed in terms of σ, τ, and f_c' as follows:

$$\frac{\tau}{f_c'} = 0.1\left[0.62 + 7.86\left(\frac{\sigma}{f_c'}\right) - 8.46\left(\frac{\sigma}{f_c'}\right)^2\right]^{1/2} \quad \ldots\ldots\ldots\ldots\ldots(13)$$

where σ and τ are normal and shearing stresses at failure, and f_c' is the nominal compressive strength of 6 x 12-in. control cylinders.

For specific states of stress this criterion gives the following values:

State of stress	σ	τ
Pure compression	f_c'	0
Pure shear	0	$0.080\,f_c'$
Pure tension	$-0.076\,f_c'$	0
"Maximum" shearing strength	$0.4\,f_c'$	$0.156\,f_c'$

Strength obtained from the proposed straight line criterion is in reasonably good agreement with the test data for the "pure" states of stress of compression and shear, and in reasonable agreement with the pure tension strength as obtained by various experimenters. The value of maximum shearing strength $\tau = 0.156\,f_c'$ when compressive stress σ is within the range of 0.3 to $0.6\,f_c'$ is quite conservative. Test results indicate that shearing strength of $0.2\,f_c'$ can be developed under these conditions.

The limitations of the proposed criteria are as follows: strength is defined as maximum load carried by the test specimen; criteria are applicable to biaxial states of stress varying from pure shear to pure compression; criteria are not applicable to biaxial compression states of stress or to triaxial states of stress; duration of loading to failure is approximately 5 to 10 min; concrete is in a relatively wet condition; the physical characteristics of the ingredients (cement, sand, and coarse aggregate) and the mix proportions are similar to those used in this study. Within the limitations of this investigation the proposed criterion is in good agreement with test results.

APPLICATIONS OF FAILURE CRITERION

To determine strength of reinforced concrete structures under various conditions of loading and environment on a rational basis, the following problems must be considered: (a) stress distribution in uncracked and cracked reinforced concrete structures and (b) criteria of failure of plain concrete under various conditions of stress and environment. Solution of these problems would lead to the clarification of the mechanism of crack propagation and failure in concrete structures.

Determination of stresses around cracks in concrete and in the reinforcement of concrete structural elements cannot be accomplished by the conventional flexure theory and very little work has been done to define these more accurately. Westergaard's work[6] is limited to stresses around a vertical crack due to pure flexure of beams, Wastlund's work[7] dealing with cracking of reinforced concrete beams is not primarily concerned with stress distributions, and Saliger's work[8] deals primarily with bond stresses. If the stresses in reinforced concrete elements could be defined with an adequate measure of precision then failure criteria developed earlier in the paper could be applied to define conditions governing crack propagation and ultimate collapse of the structure.

As an indication of the possibility of applying the failure criteria developed in this study, a new method of determining the ultimate shear strength of reinforced concrete beams without web reinforcement is proposed. This method is limited to rectangular beams subjected to concentrated loads, and so proportioned that the shearing failure loads approach the flexural capacity of the beam. Beams so proportioned would seem to define a reasonably balanced design, as beams without web reinforcement having flexural capacities greatly in excess of shearing strength are evidently overreinforced in flexure or perhaps more precisely underreinforced in shear.

To simplify the determination of the ultimate shearing strength, the following conventional assumptions are made:

1. Concrete cannot resist tension;
2. Tension reinforcement does not transfer shear; and
3. Failure occurs by the destruction of the compression zone.

The stresses in the shear-compression zone at loads approaching the ultimate vary throughout the depth kd of this zone (Fig. 8). The compressive and shearing stresses at a given point in concrete are σ and τ, respectively, and the stress in the steel reinforcement at the critical crack is f_s.

Assuming that the stresses σ, τ, and f_s can be defined in terms of external forces, geometry of cross section, rate of loading, and material properties, and that the failure criterion $\tau = f(\sigma)$ is known, three hypotheses of failure are possible.

1. It is possible to assume that failure occurs when σ and τ at some critical point satisfy the failure criterion. In this case the criterion $\tau = f(\sigma)$, may be dependent on the nature of stress distribution, and the critical point would be defined for particular stress distribution.

2. It is also possible to assume that ultimate failure occurs when σ and τ satisfy the failure criterion at all points. In this case, the failure criterion may also be dependent on the nature of stress distribution.

3. Finally, a simple assumption can be made that failure occurs when the average stresses $\bar{\sigma}$ and $\bar{\tau}$ satisfy the failure criterion $\bar{\tau} = f(\bar{\sigma})$, where $\bar{\sigma}$ and $\bar{\tau}$ are defined as:

$$\bar{\tau} = \frac{V_u}{A_e} \quad \ldots\ldots (14)$$

and

$$\bar{\sigma} = \frac{C}{A_e} \quad \ldots\ldots (15)$$

where V_u is the ultimate shear on the section, A_e = portion of the cross section effective in resisting shear V_u and compression force C, and C is the compression force due to moment on the section, such that $C = M/jd$ (Fig. 8).

If the average normal stress $\bar{\sigma}$ at loads approaching failure is known, then $\bar{\tau}$ can be determined from failure criterion $\bar{\tau} = f(\bar{\sigma})$ defined by Eq.(13), and the shearing strength V_u can be determined from Eq.(14), if the effective area A_e is known.

Determination of $\bar{\sigma}$ and A_e depends on the mechanism of failure. Two types of failures have been observed in beams failing in shear. The so-called diagonal tension failure is characterized by formation of a large diagonal crack followed almost immediately by crushing of the concrete in the com-

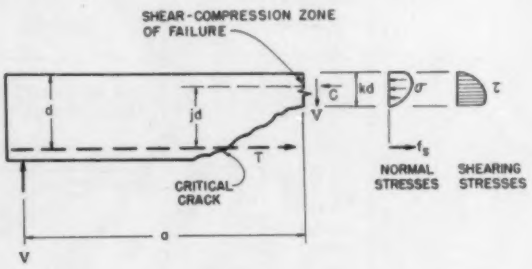

Fig. 8—Shear failure in a simply supported beam

TABLE 8—SELECTED BEAM TEST DATA, SHEAR-COMPRESSION FAILURE*

Beam	f_c' psi	f_y, psi	p	d, in.	a/d	b, in.	V_t, kips	pf_y, psi	$\frac{\sigma}{f_c'}$	k	$\frac{\tau}{f_c'}$	V_u, kips	$\frac{V_u}{V_t}$
Tests by Gaston, 1952, Reference 11 Loads at third points Age at test about 28 days													
T2Ma	4320	47,700	0.0138	10.6	3.4	6	9.25	658	0.72	0.212	0.139	8.0	0.87
T2Mb	4020	48,300	0.0138	10.6	3.4	6	9.75	666	0.74	0.224	0.135	7.7	0.79
T2Mc	4470	46,800	0.0190	10.6	3.4	6	12.50	889	0.71	0.280	0.140	11.1	0.89
Tests by Laupa, 1953, Reference 11 Center load through 6 x 12-in. column stub Age at test about 28 days													
S-2	3900	41,200	0.0208	10.6	4.54	6	9.5	857	0.74	0.297	0.135	9.9	1.04
S-3	4690	59,400	0.0252	10.4	4.60	6	11.9	1497	0.70	0.458	0.143	19.2	1.61
S-4	4470	44,800	0.0321	10.4	4.63	6	12.5	1438	0.71	0.456	0.141	17.9	1.43
S-5	4330	45,700	0.0411	10.3	4.66	6	11.2	1878	0.72	0.604	0.140	22.7	2.03
S-11	2140	47,500	0.0190	10.5	4.57	6	7.6	903	0.88	0.481	0.093	6.1	0.80
S-13	3800	44,100	0.0411	10.3	4.66	6	11.2	1813	0.75	0.638	0.133	20.0	1.79
S-1	3940	44,600	0.0146	10.7	4.79	6	8.4	651	0.74	0.224	0.135	7.6	0.91
S-9	2140	44,300	0.0093	10.7	4.48	6	5.8	411	0.88	0.220	0.093	2.8	0.49
S-10	2280	44,800	0.0139	10.6	4.54	6	7.7	623	0.86	0.317	0.092	4.2	0.55
Tests by Moody, Series A, 1953, Reference 9 Center load at midspan Age at test about 28 days													
A 1	4400	45,000	0.0217	10.3	3.06	7	13.5	976	0.71	0.312	0.140	13.9	1.03
2	4500	45,000	0.0215	10.5	3.00	7	15.0	967	0.71	0.304	0.140	14.1	0.94
3	4500	45,000	0.0222	10.6	2.99	7	17.0	990	0.71	0.314	0.140	14.6	0.86
4	4570	45,000	0.0237	10.6	2.96	7	16.0	1080	0.70	0.332	0.141	15.9	0.99
B 1	3060	45,000	0.0162	10.5	3.00	7	12.6	720	0.80	0.298	0.120	8.1	0.64
2	3125	45,000	0.0163	10.6	2.99	7	13.5	720	0.79	0.296	0.120	8.2	0.61
3	2785	45,000	0.0160	10.6	2.96	7	12.5	720	0.82	0.317	0.113	7.4	0.59
4	2430	45,000	0.0166	10.7	2.95	7	12.5	720	0.85	0.362	0.102	6.7	0.54
C 1	920	45,000	0.0081	10.6	2.99	7	4.5	365	1.14	0.348	—	—	—
2	880	45,000	0.0083	10.7	2.94	7	5.5	373	1.20	0.354	—	—	—
3	1000	45,000	0.0080	10.7	2.93	7	5.7	360	1.00	0.360	—	—	—
4	980	45,000	0.0082	10.8	2.92	7	5.6	369	1.05	0.359	—	—	—

Tests by Moody, Series B, 1953, Reference 9
Loads at third points
Age at test about 28 days

1	5320	45,000	0.019	10.6	36	6	13.0	855	0.67	0.240	0.147	12.3	0.95
2	2420	45,000	0.019	10.6	36	6	8.0	855	0.85	0.417	0.103	6.6	0.82
3	3730	45,000	0.019	10.6	36	6	11.7	855	0.75	0.305	0.132	9.5	0.81
4	2230	45,000	0.019	10.6	36	6	9.9	855	0.87	0.442	0.095	5.9	0.60
5	4450	45,000	0.019	10.6	36	6	11.7	855	0.71	0.271	0.140	10.7	0.92
6	2290	45,000	0.019	10.6	36	6	7.9	855	0.86	0.433	0.098	6.1	0.88
7	4480	45,000	0.019	10.6	36	6	11.5	855	0.71	0.269	0.141	10.7	0.93
8	1770	45,000	0.019	10.6	36	6	7.0	855	0.92	0.528	0.075	2.9	0.42
9	5970	45,000	0.019	10.6	36	6	12.0	855	0.66	0.219	0.150	12.4	1.04
10	3470	45,000	0.019	10.6	36	6	11.0	855	0.77	0.321	0.128	9.3	0.84
11	5530	45,000	0.019	10.6	36	6	13.5	855	0.66	0.234	0.149	12.2	0.91
12	2925	45,000	0.019	10.6	36	6	10.6	855	0.81	0.363	0.116	7.8	0.74
13	5480	45,000	0.019	10.6	36	6	12.5	855	0.66	0.235	0.149	12.2	0.98
14	3270	45,000	0.019	10.6	36	6	9.7	855	0.78	0.334	0.123	8.5	0.88
15	5420	45,000	0.019	10.6	36	6	11.5	855	0.67	0.238	0.148	12.1	1.05
16	2370	45,000	0.019	10.6	36	6	8.5	855	0.85	0.422	0.100	6.3	0.75

Tests by Viest and Morrow, 1955, Reference 10
Center load through 12 x 14-in. column stub
Age at test about 28 days

B40B4	5040	54,800	0.0185	14.5	2.76	12	35.0	1380	0.68	0.296	0.145	37.6	1.07
B56B2	2130	68,300	0.0185	14.5	3.86	12	22.5	1260	0.88	0.671	0.090	22.6	1.01
E2	2130	67,000	0.0058	14.5	3.86	12	17.9	390	0.88	0.207	0.090	6.9	0.39
A4	3620	47,800	0.0241	14.8	3.80	12	31.0	1150	0.76	0.419	0.130	35.0	1.13
B4	3950	63,900	0.0185	14.5	3.86	12	27.5	1180	0.74	0.404	0.135	37.6	1.37
E4	4120	62,200	0.0124	14.5	3.86	12	24.5	770	0.73	0.256	0.136	24.9	1.02
A6	5780	63,600	0.0379	14.0	4.00	12	40.0	2410	0.66	0.632	0.148	40.8	2.27
B6	6630	67,600	0.0183	14.6	3.82	12	30.7	1240	0.63	0.285	0.150	49.8	1.62
B70B2	2370	67,000	0.0186	14.4	4.87	12	20.0	1250	0.86	0.611	0.098	24.6	1.23
A4	3950	63,200	0.0246	14.5	4.83	12	29.8	1550	0.74	0.532	0.134	49.1	1.65
A6	6520	63,100	0.0383	14.0	5.00	12	40.0	2410	0.63	0.567	0.150	92.8	2.32
B84B4	3950	67,400	0.0188	14.3	5.86	12	25.0	1270	0.74	0.433	0.134	39.4	1.58
B4R	4160	50,000	0.0186	14.5	7.79	12	19.0	930	0.73	0.307	0.136	30.2	1.59

*NOTE: The author makes no differentiation between diagonal tension failures and shear-compression failures.

pression zone. This type of failure occurs in beams of intermediate length. The so-called shear-compression failure is characterized by formation of a diagonal crack which gradually flattens out, extends into compression zone, and finally causes crushing of concrete. This type of failure usually occurs in relatively short beams.

It has been observed that for beams considered here ultimate shearing failure occurs in the compression zone at some undetermined section a distance a from the support (Fig. 8). Furthermore, the mechanism of failure[9,10,11] for the class of beams described above is such that the following assumptions appear to be reasonable:

1. The longitudinal reinforcement stress f_s at the critical crack is approximately equal to the yield strength f_y of the reinforcement. This assumption appears to be valid for beams with moderate amounts of longitudinal reinforcement, $0.02 > p > 0.01$, made of mild steel with a yield point stress $f_y < 50$ ksi, and with a large yield-strain range. For beams with small amounts of reinforcement the stress f_s may be in a strain-hardening region where f_s is greater than f_y, and for beams with high percentage of reinforcement, or reinforced with steel which has no well defined yield strength, the stress f_s may be indeterminate, i.e., f_s may be greater or less than f_y.
2. Average stress $\bar{\sigma}$ in the compression zone is approximately equal to the compression stress developed in plain concrete prisms failing in flexure. Recent experimental studies[12] indicate that this can be approximated by

$$\bar{\sigma} = \frac{3900 + 0.35 f_c'}{3200 + f_c'} f_c' \quad \ldots\ldots (16)$$

This assumption appears to be valid for beams with concrete strength greater than 3000 psi. For low values of f_c' the value of $\bar{\sigma}$ may exceed f_c' hence Eq. (16) does not appear to be a reasonable approximation for $\bar{\sigma}$ when f_c' is less than 3000 psi.

The effective area A_c of the shear compression zone for rectangular beams is equal to bkd, and can be determined by setting $C = T$, as follows:

$$C = \bar{\sigma} A_c = \bar{\sigma} bkd = T = f_s A_s = f_y pbd \quad \ldots\ldots (17)$$

Therefore

$$k = \frac{p f_y bd}{\bar{\sigma} bd} = \frac{p f_y}{\bar{\sigma}} \quad \ldots\ldots (18)$$

where $\bar{\sigma}$ is given by Eq. (16).

The ultimate shearing strength V_u is obtained from Eq.(14) as follows:

$$V_u = \bar{\tau} A_c = \bar{\tau} bkd \quad \ldots\ldots (19)$$

where $\bar{\tau} = f(\bar{\sigma})$ is determined either from Eq.(13) or from Fig. 7, and k is defined by Eq.(18).

This procedure was used to calculate values of shearing strength V_u for 54 beams tested in the recent studies at the University of Illinois, as shown in Tables 8 and 9. These beams were selected on the basis of sufficiently

large span/depth ratios so that the beams exhibited typical shear compression failures. It can be seen from the tables that the method is valid for beams having concrete strength f_c' greater than 3000 psi, and percent and type of reinforcement characterized by values of pf_y in the range of 600 to 1000. Good agreement is obtained for all of the 21 beams which meet the above limitations. The remaining beams fall outside the range of validity of the method proposed here, either because of amount of reinforcement or low concrete strength. For the 21 beams for which the method of analysis applies, the average ratio of calculated shear strength to that observed in the test is 0.90, and the range of 90 percent of the values is 0.79 to 1.05.

TABLE 9—SELECTED BEAM TEST DATA*

Beam	pf_y, psi	f_c', psi	$\frac{V_u}{V_t}$	$\frac{V_f}{V_t}$
T2Ma	660	4320	0.87	1.20
T2Mb	670	4020	0.79	1.15
T2Mc	890	4470	0.89	1.17
S-1	650	3940	0.90	1.02
S-2	860	3900	1.04	1.09
A-1	980	4400	1.03	1.47
A-2	970	4500	0.94	1.36
A-3	990	4500	0.86	1.26
B-1	720	3060	0.64	1.22
B-2	770	3125	0.61	1.16
1	860	5320	0.95	1.04
3	860	3470	0.81	1.11
5	860	4450	0.92	1.14
7	860	4480	0.93	1.16
9	860	5970	1.04	1.00
10	860	3470	0.84	1.17
11	860	5530	0.91	1.01
13	860	5480	0.98	1.08
14	860	3270	0.88	1.33
15	860	5420	1.05	1.18
B56E4	770	4120	1.02	1.25
Average			0.90	1.22

*Refer to Table 8 for sources of data.

The shearing strength V_u defined by Eq (19) is based upon quantities $\bar{\sigma}$ and kd which for the beams considered here are derived from the assumptions that $f_s = f_y$ and $\bar{\sigma} = f(f_c')$. Thus V_u is completely defined when the proportions of the beam b, d, and p, the loading conditions, and the material characteristics f_y and f_c' are known. As expression for V_u does not contain any empirical constants derived from beam tests, the agreement between the computed values of V_u and the test values V_t serves as an independent check on the validity of the failure criterion.

It may be argued that the assumption of $f_s = f_y$ is tantamount to the assumption of flexural failure when maximum bending moment M_u is

$$M_u = Tjd = (pf_y\, bd)\, jd = pf_y\, bjd^2 \qquad (20)$$

where jd is the lever arm of the internal resisting moment as defined by Hognestad, Hansen, and McHenry.[12]

Neglecting the effect of beam weight, the shear V_f at flexural failure

$$V_f = M/a = \frac{pf_y\, bjd^2}{a} \qquad (21)$$

where a is the distance from the support to the point of load application, usually called the shear span. Values of V_f have been computed for the 21 beams and are listed in Table 9. The average value of V_f/V_t is 1.22 with a range of 1.01 to 1.47. This is significantly higher than the average V_u/V_t of 0.90 with a range of 0.79 to 1.05 (based on 90 percent of the values), in-

dicating that for the 21 beams considered here the proposed method for determining shearing strength is in substantially better agreement with test results than the assumption of a flexural failure.

CONCLUDING REMARKS

Use of "mean" stresses appears to be a powerful tool for determining failure criteria. The particular criteria proposed here are valid only within the limits of state of stress, rate and duration of loading, and physical characteristics of the materials used in this investigation.

It is evident that strength of concrete is a function of the state of stress and can not be predicted by limitations of tensile, compressive, and shearing stresses independently of each other. For example, concrete having pure compressive strength of f_c', and pure shearing strength of 0.080 f_c', would fail under compressive stress of 0.5 f_c' with the shearing stress increased to approximately 0.2 f_c'. Therefore, strength of structural elements can be properly determined only by considering interaction of the various components of the state of stress. Such conditions as shrinkage, restraint to contraction or expansion, foundation settlements, duration of loading, and previous stress history, may have an important effect on the state of stress at the critical section, and must be carefully evaluated, particularly with respect to possible reduction of strength.

The method proposed for determining shearing strength of beams is applicable only to the special group defined here and is not expected to have general validity. Strength of concrete beams subjected to combined shear, flexure, and axial loads is dependent upon a large number of variables. Particularly, the problem of determining stress distribution in these beams at failure awaits a rational solution. For a class of beams considered here the stress distribution was approximated with reasonable accuracy and ultimate loads observed in tests were verified using the proposed failure criteria.

Of utmost importance in the method proposed here is the concept that the cross-sectional area effective in resisting shearing and normal stresses at a critical section is that of the uncracked zone. Thus, the conventional definition of the nominal shearing stress $v = V/bjd$, based on the nominal effective $A = bjd$ is quite unsatisfactory because the actual effective area resisting shear is more nearly equal to $A_e = bkd$.

It follows that shearing stresses at the critical section may be considerably greater than the computed nominal stresses. Assuming approximate values of k and j as $\frac{3}{8}$ and $\frac{7}{8}$, the ratio of actual shearing stresses to the nominal value may be equal to j/k or 7/3. Thus, when the calculated nominal shearing stress is equal to 0.03 f_c', the actual shearing stress may be equal to 0.07 f_c'. It is readily seen that this stress has a low factor of safety at a section where compressive stress is nearly zero, say at the point of inflection, and cannot be resisted at all at a section where slight tension may exist due to longitudinal forces induced by shrinkage, temperature variations, or other causes.

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Title No. 55-21

Economic Factors in Prestressed Lift-Slab Construction*

By EDWARD K. RICE†

SYNOPSIS

The use of prestressed concrete lift slabs to obtain substantial advantage in performance of the slab is discussed. Items covered include cost trends in conventional versus lift-slab construction and factors affecting the economy of prestressed lift-slab construction including building layout, structural framing schemes, use of precast columns, connections, and prestressing layout.

INTRODUCTION

The rapid growth in the use of prestressed concrete lift-slab construction in the southwestern part of the United States has clearly demonstrated the basic economy of this type of construction. To date, at least 41 structures involving 2,000,000 sq ft of prestressed concrete lift slabs have been built. An additional 3,000,000 sq ft of slabs are now known to be in the design or building stage. The completed structures range in size and occupancy from 10,000-sq-ft industrial buildings with roof parking to a six-story state office building of 250,000 sq ft.

A major factor in the success of prestressed concrete lift-slab construction is that the prestressing of the slabs and the lifting construction technique complement each other to provide many advantages not experienced when either is used alone. By prestressing the slabs, deflections experienced in reinforced slabs can be controlled and, by using lift-slab construction methods, the elastic and some of the plastic shortening of the prestressed slabs can conveniently be allowed for before the connection to the columns is made.

THE LIFT-SLAB DEFLECTION PROBLEM

In conventional cast-in-place reinforced concrete construction, it is common practice to camber the soffit forms sufficiently to allow for the dead-load deflection of the slabs. However, in lift-slab construction, it is generally impractical to provide sufficient camber in the grade slab, which is used as a soffit form, to allow for the dead-load deflection of a reinforced concrete slab.

On some buildings, a waste slab has been cast on top of the grade slab to provide a cambered surface. On others, the reinforced concrete lift slabs

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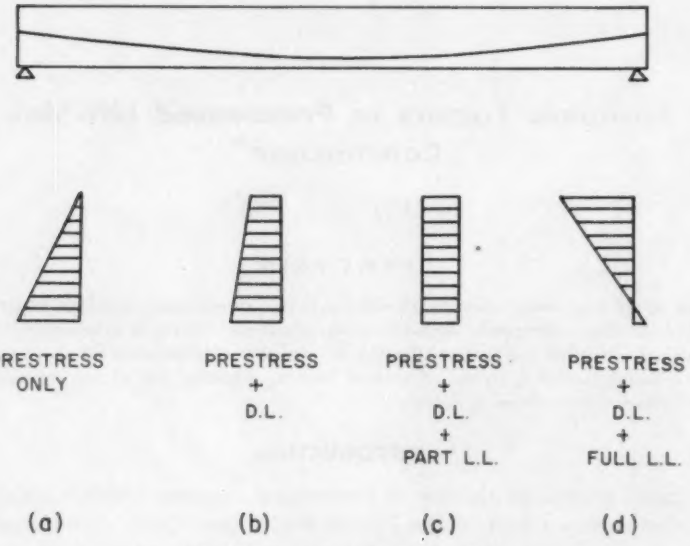

Fig. 1—Bending stress at various stages

have been made deeper and the reinforcing steel stresses reduced in an effort to minimize deflections in the slabs. These measures produce somewhat satisfactory results; however, when such measures are resorted to, some of the cost savings of lift-slab construction have been lost.

Where particular care has not been taken to limit the deflection, reinforced lift slabs have deflected as much as 2 in. immediately after lifting, and, had additional supports not been provided, would have deflected more than 3 in.

Experience indicates that only for spans of less than perhaps 18 ft are reinforced lift slabs satisfactory in behavior and economical in construction. To achieve satisfactory results economically with lift-slab spans greater than 18 ft, the slabs must generally be prestressed.

By properly placing the prestressing tendons in a slab, the dead-load deflection plus any desired amount of the live-load deflection may be balanced by the internal prestressing force. In this way, it is possible to cast a lift slab on a flat grade slab and have it remain flat after lifting.

Perhaps the simplest way of visualizing how the slab loads are balanced by prestressing force is to look at the bending stress diagrams for a simple span slab. In Fig. 1(a) is shown the stress diagram of the prestressing alone. The lower fibers at midspan are compressed. Fig. 1(b) shows the stress diagram after the slab has been lifted. In this case, the lower fibers are still compressed somewhat more than the upper fibers; this difference will cause the slab to tend to camber upward a small amount.

A good design practice is to estimate the fixed portion of the live load and to design the slab so that it will be flat when the dead load plus the fixed por-

tion of the live load is applied. This condition is shown in Fig. 1(c). In office-type buildings, it has been found by weighing the actual contents and dividing by the area that the fixed portion of the live load is only about 8 lb per sq ft.

In Fig. 1(c), the stresses in the lower fiber and the upper fiber are the same, and since the compressive shortening in the upper and lower fibers is equal there is no bending in the slab at this point. If the bending stress distribution is rectangular at every point in the slab, then the slab will be flat under the given loading condition. With a rectangular bending stress distribution in the slab, the effect of plastic flow does not cause the slab to deflect with time as a reinforced slab does. Since the stresses in the upper fiber of the slab are the same as in the lower fiber, the amount of plastic shortening will be the same along both fibers and the slab will remain flat.

When the full live load is applied to the slab, the bending stress diagram may look like Fig. 1(d). Under this condition, the slab will have some deflection; however, upon removal of the load, complete recovery should be experienced. Under full live load it is often good practice to allow a certain amount of tension in the slab. If no tension is allowed under the design live load, the slab will have excessive compression in the lower fibers. This condition will tend to camber the slab upward. The amount of allowable tension will vary depending upon the ultimate strength of the slab and other considerations.

Field measurements of deflections of prestressed flat slabs have been made both immediately after lifting and at intervals after the building was completed. These measurements have shown that properly designed prestressed slabs have little if any noticeable deflection. Similar measurements on conventionally reinforced lift slabs have shown objectionable deflections which increase with time.

COMPARISON OF COST TRENDS: CONVENTIONAL CAST-IN-PLACE CONSTRUCTION AND LIFT-SLAB CONSTRUCTION

When working with new construction methods, in which seemingly large advances in technology are being made on each project, it is of interest to compare the relative cost of conventional methods against the new methods to see if the advancing technology is really reducing the cost to the owner.

To make such a comparison, a typical building bay of 500 sq ft has been chosen. For conventional construction, a 9½-in. slab was assumed, with 6 lb of reinforcing steel per sq ft and a 12 in. square column 12 ft high. For the prestressed concrete lift-slab structure, a 6-in. slab with 1 lb of prestressing steel plus 0.5 lb of regular reinforcing steel per sq ft with either a 12 in. square precast concrete column or a standard steel wide flange column were used in estimating the material and labor. Applicable unit cost figures for the San Francisco area for years 1952 through 1957 were assembled. From these

TABLE 1—COST COMPARISON OF CONVENTIONAL CONSTRUCTION AND PRESTRESSED LIFT-SLAB CONSTRUCTION*

Item	Quantity required per sq ft of slab	1957		1954		1952	
		Unit price	Unit cost per sq ft of slab	Unit price	Unit cost per sq ft of slab	Unit price	Unit cost per sq ft of slab
Conventional construction: 9½-in. slab; 6 lb steel per sq ft; 500 sq ft per column; 12 x 12-in. column, 12 ft high							
FORMS							
Plywood	0.5 sq ft	14.91	0.075	14.91	0.075	12.30	0.062
Shores	1 fbm	83.00	0.083	62.00	0.062	76.00	0.076
Labor	0.1 man-hr	3.45	0.345	2.90	0.290	2.45	0.245
CONCRETE							
Material	0.03 cu yd	12.75	0.385	12.15	0.370	10.30	0.310
Labor	0.08 man-hr	2.42	0.200	2.075	0.166	1.70	0.136
REINFORCING							
Material	6 lb	0.091	0.550	0.0685	0.410	0.0650	0.390
Labor	0.0685 man-hr	3.50	0.240	3.075	0.210	2.70	0.185
COLUMNS							
Steel—Labor	0.043 man-hr	3.50	0.015	3.075	0.013	2.70	0.011
—Material	0.29 lb	0.091	0.027	0.0685	0.020	0.0650	0.019
Concrete—Labor	0.0033 man-hr	3.45	0.011	2.90	0.010	2.45	0.008
—Material	0.001 cu yd	12.75	0.013	12.15	0.012	10.30	0.010
			1.944		1.638		1.452
Prestressed lift-slab construction: 6-in slab; 1 lb prestressing steel and 0.5 lb mild steel per sq ft; 500 sq ft per column; 12 x 12-in. column, 12 ft high							
FORMS	(no edge forms above)						
CONCRETE							
Material	0.0185 cu yd	12.75	0.235	12.15	0.225	10.30	0.190
Labor	0.039 man-hr	2.42	0.095	2.075	0.081	1.70	0.066
REINFORCING							
Material	0.5 lb	0.091	0.045	0.0685	0.034	0.065	0.033
Labor	0.0057 man-hr	3.50	0.020	3.075	0.018	2.70	0.015
PRESTRESSING							
Material	1 lb	0.55	0.550	0.65	0.650	0.85	0.850
Labor	0.0285 man-hr	3.50	0.100	3.075	0.088	2.70	0.077
LIFTING	1 sq ft	0.25	0.25	0.30	0.300	0.35	0.350
COLLARS	1/500 sq ft	35.00	0.07	50.00	0.100	60.00	0.120
			1.365		1.496		1.701
CONCRETE COLUMNS (Same as conventional)			0.066		0.055		0.048
Miscellaneous Iron $25 each per 500 sq ft			0.050		0.040		0.040
Erection $15 per 500 sq ft			0.030		0.025		0.025
			1.511		1.616		1.814
STEEL COLUMNS	1.08 lb per sq ft	0.20	0.215	0.16	0.173	0.14	0.153
			1.580		1.669		1.854

*All costs shown in dollars.

data, shown in Table 1, a relative cost index in dollars per square foot of slab was compiled.

In Fig. 2 are shown the cost indices for the years 1952 through 1957. The rising cost curve for conventional construction is familiar to those acquainted with current construction cost trends. There are two curves for prestressed concrete lift-slab construction—one for concrete columns and the other for steel columns. It was anticipated that a greater divergence would be shown between the lift slab with concrete columns and that with steel columns. The divergence will probably be more pronounced when the 1958 points are plotted.

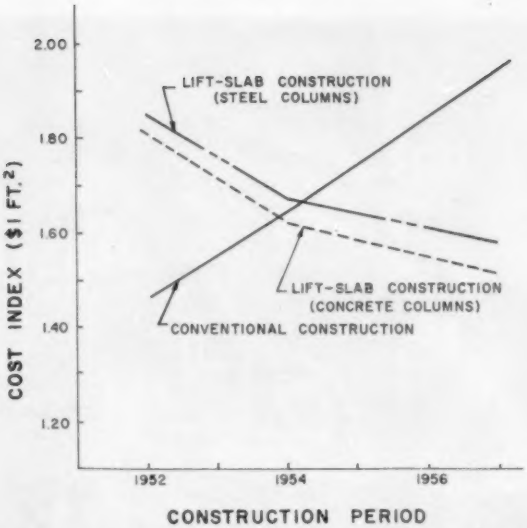

Fig. 2—Cost comparison of conventional construction and lift-slab construction

Examine the cost curves chronologically. During the period from 1952 to 1954, prestressed lift-slab construction was generally restricted to buildings in which its use was dictated by factors other than the first cost of labor and material. In some structures, prestressed lift-slab construction was used to shorten the construction time. Records show a time saving of approximately 30 percent in favor of prestressed lift-slab construction over conventional construction. Other structures in this period were of prestressed lift-slab construction because of architectural considerations.

In 1954, alternative prestressed designs were prepared for two buildings for the state of California. On these two projects, the prestressed concrete designs were bid in competition with conventional cast-in-place reinforced concrete construction and a reinforced concrete lift-slab alternative. On one building, the prestressed alternative was bid slightly cheaper than the other types of construction; on the second project, the conventionally reinforced lift slab was bid slightly cheaper. On these projects, time of construction was not a factor and the architectural requirements could be met equally well with any of the methods. After 1954, it became evident that prestressed lift-slab construction, when properly conceived and designed, was cheaper on a first cost basis than other types of construction.

FACTORS AFFECTING THE ECONOMY OF PRESTRESSED LIFT-SLAB CONSTRUCTION

Of course, not all structures can be economically framed of prestressed concrete lift-slab construction. Some types of structures which are not now readily adaptable to prestressed lift-slab construction include structures on

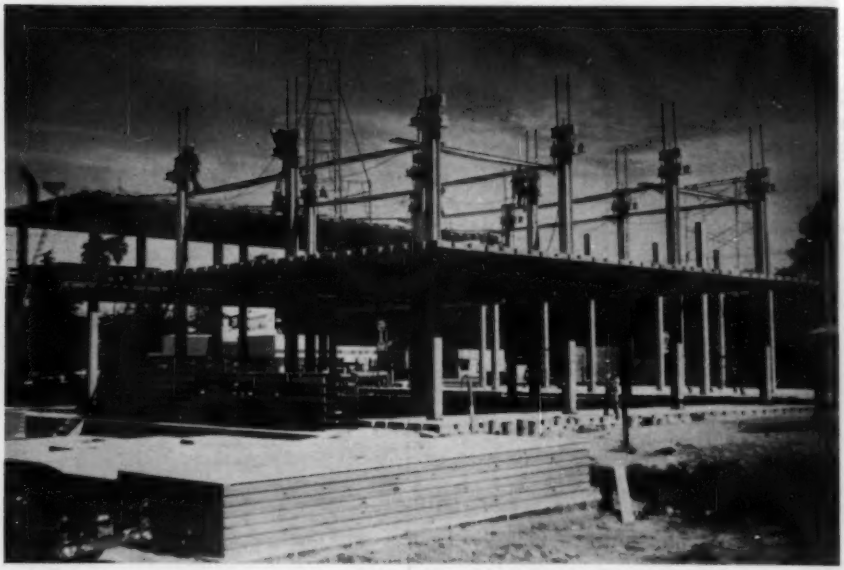

Fig. 3—A 6-in. prestressed roof slab, having been cast on top of second-floor slab at ground level, is being lifted at 15 ft per hr to top of precast concrete columns

hillsides with stepped grade slabs, and structures which have a fairly close spacing of concrete walls which may serve as bearing walls, such as prison cell blocks. Also, the expense of cutting prestressing tendons to a great number of lengths makes structures of highly irregular shapes generally uneconomical.

Considering total economy, the building layout and structural framing scheme are more critical in prestressed lift-slab work than in other types of construction.

Column layouts, stairwell and plumbing chase locations, size and shape of sections to be lifted, and column connections are all factors which must be carefully considered. As in most new types of construction, lift-slab construction often requires a greater effort on the part of the designer than is required for more conventional construction. To achieve maximum economy, careful layout of practical details is a "must," and in order to assure a satisfactory construction, a high degree of job site supervision is required.

Structural framing schemes

Several structural framing schemes have been used including the following:

Solid flat slabs—These are the simplest and easiest to design and construct. Solid slabs are generally economical for spans of 20 to 30 ft. Slab thicknesses of 4½ to 9 in. have been used. At the present, most roof slabs are either 5½ or 6 in. thick and floor slabs 6 or 6½ in. thick. The structure shown in Fig. 3 is a furniture warehouse in Los Angeles. The second-floor slab is 6½ in.

Fig. 4—Post-tensioned prestressing tendons in a 6½-in. two-way prestressed flat floor slab. Each tendon contains 7 wires of ¼ in. diameter high tensile steel

thick, has a 23-ft span, and is designed for a live load of 100 psf. Two bays of this building were load tested with sand to a live load of 150 psf. The maximum deflection was less than 3/16 in. After 24 hr, the load was removed and the slab returned to its original position. Solid flat slabs have unusually large factors of safety. Shown in Fig. 4 are the prestressing tendons in place in a two-way prestressed slab. In a vertical plane, the tendons are curved to follow generally the inverse of the bending moment curve.

Inverted beams—For long roof spans in one direction, inverted beams on the roof are a practical framing scheme. Spans of 40 to 140 ft parallel to the beams

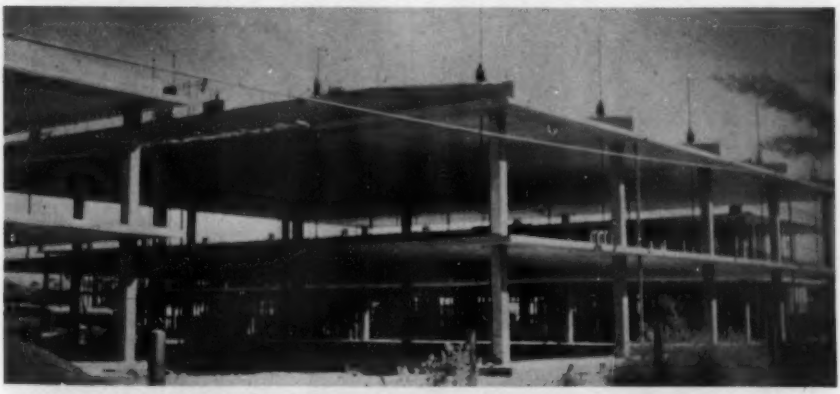

Fig. 5—Clear areas on the ground floor are made practicable by hanging the second floor from inverted beams on the roof

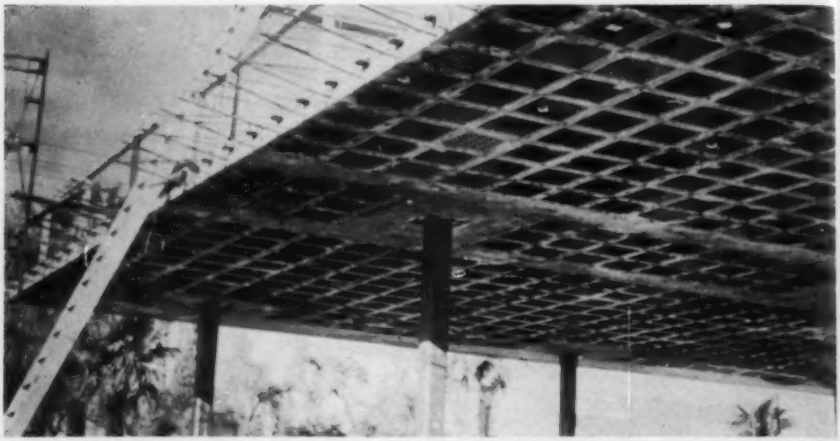

Fig. 6—A prestressed concrete cored lift slab is used to achieve long spans in a new classroom building at San Jose State College

with column spacings between the beams of 20 to 24 ft are economical. Both hollow box, I-beam, and rectangular cross sections have been used.

For some buildings requiring clear areas on the ground floor, the second floor has been hung from the inverted roof beams with tie rods (Fig. 5).

Cored or waffle slabs—When large bays are required, cardboard boxes are used to core the slab and thereby to reduce the dead load (Fig. 6). Bays of 30 to 50 ft are economical in cored-slab construction. Since the concrete stresses are much higher in cored slabs, the designer and the constructor must execute a cored-slab structure more carefully than flat-slab construction.

Fig. 7—In one direction the prestressed concrete lift-slab span may be increased by forming "dropped" beams into the grade slab. Parking deck spans of 52 ft with an 18-in. total structural depth have been constructed

Dropped beams—When flat roof areas are required for roof parking, long spans in one direction can be achieved by a "dropped beam" slab configuration, with the beams spanning between columns and a one-way slab spanning between beams. The beam is easily formed by omitting short strips of the grade slab and placing the beam forms in the resulting trench. The slab and the "beam" are cast at one time resulting in a monolithic section (Fig. 7). Clear spans of 52 ft with a total structural depth of 18 in. have been designed to carry a parking deck load.

Precast concrete columns

The use of precast columns in lift-slab construction has helped reduce the over-all cost. The pull-rod spacing of early lifting equipment restricted the column size to 10½ in., which was generally not wide enough for practical concrete columns. Later equipment has wider rod spacing which allows columns up to 22 in. wide to be used.

Concrete columns, in general, have a much larger moment of inertia than do equivalent steel columns. This increase in bending resistance adds to the stability of the columns during the lifting of the slabs. Often the temporary bracing, which would be required if steel columns were used, can be eliminated when concrete columns are used.

The base connection between the column and the grade slab is critical in lift-slab construction as the column must act as a cantilever during erection

Fig. 8 (left)—A precast concrete column is placed into a preformed socket in the grade slab. After the column is plumbed, the space between the column and the socket will be dry-packed. The column extends into the socket 18 in. Fig. 9 (right)—By casting steel angles into the precast concrete columns, a standard double nut column-to-footing connection may be used

of the slabs. In general, two types of base connection have been used. Shown in Fig. 8 is a socket type of connection in which the column is dry-packed into a preformed socket in the grade slab. In Fig. 9, an anchor bolt connection to the footing is shown. Angles have been cast in the column to form a base plate which is secured to the footing with conventional anchor bolts and double nuts.

When long columns are required, they can be more economically built of prestressed concrete. Prestressing tends to increase the elastic stability of the columns.

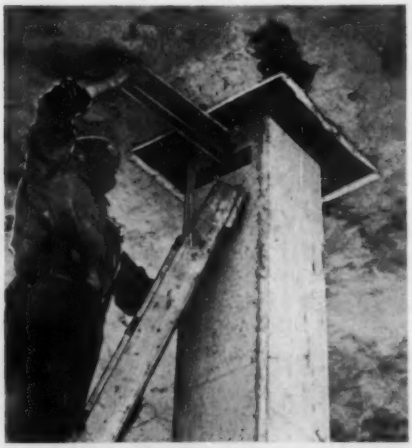

Fig. 10—The workman is sliding a 2 x 4 x 16-in. steel shear key into a preformed slot in the concrete column. After the key is in place, the lift slab, which has been raised slightly high, will be lowered onto the key to make the final connection between the column and the lift slab. Wind and earthquake loads are taken by shear walls

Column-to-slab connection

The connection between the column and a lifted slab is one requiring careful study. To reduce the lifting cost, the connection must be capable of rapid execution, as the lifting equipment can not be moved to the next section until the connections are made. On early work, welded connections were used which required several hours of work to complete. Recently, a shear key type of connection has been developed which can be installed in only a few minutes.

Shown in Fig. 10 is a shear key being inserted into a pocket in the precast column. When keys have been inserted through all the columns, the slab will be lowered onto the keys to make the final connection between the slab and the columns.

Lifting collars

One of the major cost factors in early lift-slab construction was the high cost of the steel lifting collars or shear collars. The prestressing of the slabs has increased the reliability and the magnitude of the shear resistance of collars.

Concrete

Both normal weight and lightweight concrete have been used in prestressed lift-slab construction. Generally a 28-day cylinder strength of 4000 psi is used, and the slabs are stressed when the concrete strength reaches 3500 psi.

A maximum aggregate size of 1½ in. is generally used to reduce the cement content and the shrinkage of the concrete.

Prestressing

To date, nearly all lift slabs have been prestressed with some type of nonbonded tendon with button head type of end anchorage. Grouted tendons have been tried, only occasionally, yielding inconsistent and often poor results.

Watertight slabs without roofing or membrane

One of the biggest advances made to date in roof construction, particularly for roof parking decks, has been in the development of prestressed concrete decks which are watertight without conventional roofing or a membrane of any type. To date, at least six prestressed concrete structures involving about 200,000 sq ft of roof slabs have been constructed without roofing.

The cost and weight savings, by eliminating the conventional roofing and wearing surface, have made roof parking an economical solution to the parking problem in areas where one-story structures are constructed.

Wall material

Various wall materials have been employed with prestressed lift-slab construction including glass, metal, masonry, and tilt-up panels.

It is evident that with rising material and labor costs and a desire on the part of owners for faster construction, there will be increasing use of prestressed lift-slab construction.

ACKNOWLEDGMENTS

Credit is due the State of California Division of Architecture for pioneering efforts in developing satisfactory and economical prestressed lift-slab construction. Anson Boyd is state architect; Charles Peterson is principal structural engineer; Allan H. Brownfield serves as supervising structural engineer.

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Title No. 55-22

W/C and compaction featured in the third of four parts

Origin, Evolution, and Effects of the Air Void System in Concrete. Part 3—Influence of Water-Cement Ratio and Compaction*

By JAMES E. BACKSTROM,† RICHARD W. BURROWS,‡ RICHARD C. MIELENZ,§ and VLADIMIR E. WOLKODOFF**

SYNOPSIS

Size distribution, frequency of air voids, spacing factor, and freezing and thawing resistance of concrete are influenced by many factors, among the most significant being water-cement ratio and degree of compaction. Increased freezing and thawing resistance generally reflects a reduction in void size and spacing factor. Such reductions are obtained, other factors being equal, through reduced water-cement ratio, increased amount of air-entraining agent, and in the case of void size through increased periods of vibration. Reduction of water-cement ratio increases the proportion of air-entraining agent necessary to produce a given air content but the air content required for maximum durability is decreased as the water-cement ratio is decreased. Increasing periods of vibration reduce the total air content and increase the specific surface of air voids, but have relatively little effect on spacing factor. For any one concrete there is an optimum air content and void spacing factor for optimum resistance to freezing and thawing. Spacing factor which obtains at optimum freezing and thawing resistance of a single concrete variously vibrated may or may not be the smallest in magnitude.

INTRODUCTION

The first report†† of this series discussed various factors influencing the development, during mixing, and the subsequent behavior of an air void system in concrete up to the point at which hardening prevents further activity of the bubbles. Part 2‡‡ discussed the various void systems developed by different types and proportions of air-entraining agents and their influence on freezing and thawing resistance. Part 3 presents results of various investiga-

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††Mielenz, Richard C., Wolkodoff, Vladimir E., Backstrom, James E., and Flack, Harry L., "Origin, Evolution, and Effects of the Air Void System in Concrete. Part 1—Entrained Air in Unhardened Concrete," ACI JOURNAL, July 1958, *Proc.* V. 55, pp. 95-122.

‡‡Backstrom, James E., Burrows, Richard W., Mielenz, Richard C., and Wolkodoff, Vladimir E., "Origin, Evolution and Effects of the Air Void System in Concrete. Part 2—Influence of Type and Amount of Air-Entraining Agent," ACI JOURNAL, Aug. 1958, *Proc.* V. 55, pp. 261-272.

tions into the effects of water-cement ratio and compaction by vibration on air void parameters and freezing and thawing resistance of concrete.

INFLUENCE OF WATER-CEMENT RATIO

If other conditions and relationships are constant, including the ratio of the air-entraining agent to mixing water, water-cement ratio influences the size distribution of air voids because the viscosity of the water phase and the air content of the cement paste are altered greatly as water-cement ratio changes. As the water-cement ratio decreases, the viscosity of the water phase increases and the air content of the cement paste decreases. The rate of diffusion of dissolved air tends to decrease with increased viscosity of the water phase and the amount of air diffused from small bubbles to large decreases with increased distance between bubbles. Hence, both factors favor retention of large values of α (specific surface of voids) at low water-cement ratio. At any water-cement ratio within practical limits, $\bar{L}$ (spacing factor) may be small or large depending upon the efficiency and concentration of the air-entraining agent employed. However, decreased viscosity and increased air content of the cement paste attending an increase in water-cement ratio

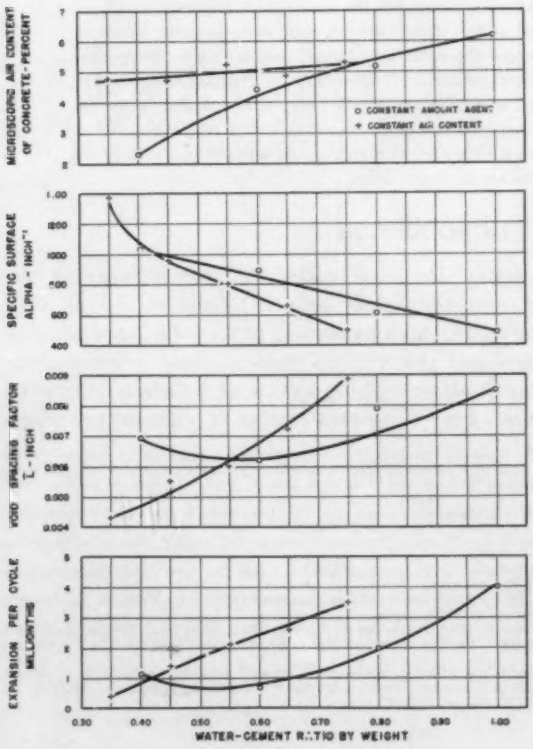
MICROSCOPIC AIR CONTENT OF CONCRETE - PERCENT
o CONSTANT AMOUNT AGENT
+ CONSTANT AIR CONTENT
SPECIFIC SURFACE ALPHA - INCH⁻¹
VOID SPACING FACTOR L̄ - INCH
EXPANSION PER CYCLE MILLIONTHS
WATER-CEMENT RATIO BY WEIGHT

Fig. 1—Effects of water-cement ratio on parameters of air void systems and freezing and thawing resistance of concrete containing either a constant amount of air-entraining agent or a constant amount of air

TABLE 1—EFFECT OF WATER-CEMENT RATIO ON THE VOID SYSTEM OF CONCRETE

Water-cement ratio by weight	Proportion of air-entraining admixture (J) ml per cu yd	Water content, lb per cu yd	Slump, in.	Paste content, percent,	Void parameters determined microscopically					Freeze-thaw expansion per cycle, millionths
					Air content, percent	Chord intercept, in.	Specific surface, $in.^{-1}$	Voids intercepted per in.	Spacing factor, $\bar{L}$, in.	
Constant amount of agent series*										
0.40	400	212	1.5	30.1	2.3	0.0039	1030	5.93	0.0069	1.1
0.60	400	208	3.0	31.9	4.4	0.0045	889	9.78	0.0062	0.7
0.80	400	207	3.0	28.1	5.2	0.0066	606	7.88	0.0079	2.0
1.00	400	194	3.0	25.6	6.2	0.0082	488	7.56	0.0085	4.0
Constant air series										
0.35	2250	365	2.5	41.2	4.8	0.0029	1380	16.56	0.0043	0.4
0.45	980	312	3.3	31.5	4.7	0.0042	952	11.18	0.0055	1.4
0.55	609	288	2.8	26.9	5.2	0.0052	800	10.40	0.0060	2.1
0.65	451	287	2.9	25.3	4.9	0.0061	656	8.04	0.0072	2.6
0.75	404	286	2.8	24.1	5.3	0.0080	500	6.63	0.0089	3.5

*Concrete wet-screened from 1½-in. maximum size aggregate to ¾-in. maximum size. Data obtained on 3 x 6-in. cylinders.

tend to decrease α, but $\bar{L}$ may still be small or large. In addition, any factor which increases the rate of hardening of concrete will tend to increase $\bar{L}$ because greater time is allowed for transmission of air from small bubbles to large.*

To illustrate, in Fig. 1 are plotted the air void parameters (Table 1) of two series of concrete mixtures in one of which the proportion of air-entraining agent was held constant;[22] in the other[23] the air content was held constant. Water-cement ratio varied widely in both series. A study of these curves indicates that an increase of water-cement ratio with amount of agent held constant tends to increase the size of the bubbles (decrease the magnitude of α) and to increase the air content of the cement paste greatly. In the usual range of mixtures, if the ratio of air-entraining agent to mixing water is maintained approximately constant as water-cement ratio is increased, the spacing factor $\bar{L}$ remains essentially constant or may decrease slightly because of the great increase in air content of the cement paste. However, if the concentration of air-entraining agent is decreased so as to maintain the concrete's air content at about the level obtained in similar concrete at lower water-cement ratio, $\bar{L}$ will increase materially with consequent decrease in resistance of concrete to freezing and thawing.

In these two series, insufficient air was entrained to provide maximum resistance to freezing and thawing for any of the concretes. In other tests, it has been found[20] that concretes containing ¾-in. maximum-size aggregate and having water-cement ratio of 0.45 and 0.55 require from 6 to 8 and from 9 to 13 percent of air to develop this maximum, and lower or higher water-cement ratio requires less or more air, respectively. Note that for air contents

*The tendency for increased $\bar{L}$ with increase in time of hardening may be more than offset in particular structures because of secondary effects of the phenomena causing the retardation, such as reduced temperature or a chemical retarder.

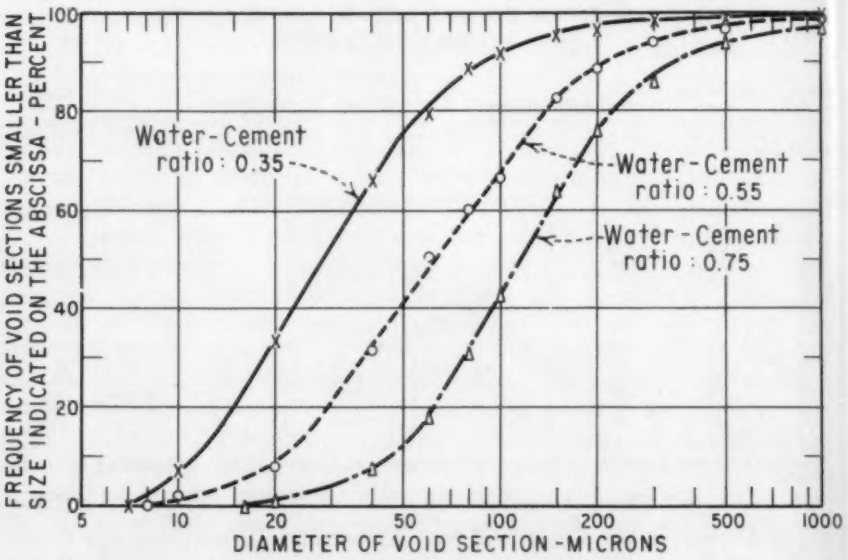

Fig. 2—Change of distribution of void section size with change of water-cement ratio at similar air content

up to about 25 percent, the two concretes were more resistant than control concretes which contained no entrained air. This excessive air content, of course, results in excessive loss of compressive strength and may have no practical application. However, it emphasizes the need for adequate air initially entrained in order to develop an effective air void system and freezing and thawing resistance.

Void size distribution

Void section distribution analyses based upon areal traverse of concrete of similar air content but differing water-cement ratio reveal widely differing frequency distribution of the void section sizes (Fig. 2). Approximately 75 percent of the void sections are 50 microns or less in diameter for the concrete of low water-cement ratio (0.35), whereas only about 12 percent of the voids are in this size range in the concrete of high water-cement ratio (0.75). The smallest void section observed in the concretes whose water-cement ratio is 0.35 or 0.55 is between 7 and 8 microns, whereas the smallest void section observed in the concrete whose water-cement ratio is 0.75 is 16 microns in diameter.

The change in void size distribution is indicated more quantitatively by the calculation devised by Lord and Willis[18] for data obtained by the modified linear traverse (Table 2). With increase in water-cement ratio from 0.35 to 0.55, the number of voids per cu in. of concrete decreases from about 7,150,000 to about 2,470,000. At a water-cement ratio of 0.75 the concrete contains

only about 700,000 voids per cu in. Also, voids less than 20 microns in diameter constitute 58 percent of the total number of voids at a water-cement ratio of 0.35 and only 9 percent of the voids at a water-cement ratio of 0.75. With water-cement ratio of 0.35 and 0.55, most of the voids are less than 20 microns in diameter, whereas at a water-cement ratio of 0.75, most of the voids are between 20 and 40 microns in diameter.

Spacing factor related to freezing and thawing resistance

Fig. 1 shows the correlation existing between the spacing factor and the freezing and thawing test results obtained from the same series of concretes cited above. With but one exception low water-cement ratio concretes possessed a smaller spacing factor and were more resistant to freezing and thawing action than the concretes of high water-cement ratio. This is true regardless of whether the air content or the amount of agent is held constant. The one exception exists in the "constant agent" series in which concrete having a water-cement ratio of 0.60 developed greater resistance than did similar concrete of 0.40 water-cement ratio. This can possibly be explained by the relatively low air content, 2.3 percent as opposed to 4.4 percent, of the richer concrete which would normally require 4 to 5 percent air for optimum durability or spacing factor. The leaner concrete, entraining more nearly its optimum percentage of air, possessed a superior spacing factor and proved to be more durable. This is not inconsistent with the findings reported previously.[20] However, this apparent reversal should not be regarded as typical. This statement is substantiated by voluminous data, not presented here because of the lack of information concerning void parameters, which show each water-cement ratio to have an individual optimum air content, with respect to freezing and thawing resistance, at which richer mixes are invariably more durable than are the leaner ones.

It must be pointed out that the foregoing analysis was made on the basis of freezing and thawing expansion, which is indicative of *internal* damage. From the standpoint of surface *scaling* and *crumbling* during freezing and thawing, the leaner mixes were also inferior but without exception, the anomaly discussed above not being apparent when weight loss is the criterion of freezing and thawing resistance.

TABLE 2—EFFECT OF WATER-CEMENT RATIO ON THE SIZE DISTRIBUTION OF VOIDS IN CONCRETE OF NEARLY CONSTANT AIR CONTENT*

Diameter of voids, microns	Number of voids per cu cm of concrete†		
	$W/C = 0.35$	$W/C = 0.55$	$W/C = 0.75$
20	255,234	73,240	4000
40	128,386	39,685	20,661
60	26,035	16,285	7878
80	13,183	8191	3234
100	7348	5215	1589
120	1847	4002	1103
140	1354	889	694
160	812	832	531
180	639	523	779
200	1271	1381	1672
250	118	129	196
300	33.7	120	136
350	15.3	61.1	118
400	59.8	94.2	57.9
450	47.7	54.1	31.8
500	68.4	104.0	79.5
600	4.3	14.9	18.2
700	3.1	17.8	17.8
800	20.3	1.6	1.7
900	—	1.3	1.3
1000	—	9.7	8.7
2000	—	1.0	2.0
3000	—	—	—
4000	—	—	—
Total	436,479.6‡	150,851.7‡	42,810.9‡

*See Table 1 for mixture proportions.
†Calculated in accordance with the method established by Lord and Willis.[16]
‡Corresponding numbers of voids per cu in. of concrete are 7,153,890, 2,472,447, and 701,655, respectively.

INFLUENCE OF COMPACTION

The method and extent of compaction used to place concrete exert a fundamental influence on the air voids. Air content is reduced by the process of compaction, primarily because movement of air bubbles to and escape from the surfaces of the concrete is facilitated by vibration, and, to a lesser extent, because some of the larger bubbles are broken into smaller bubbles in which the air content is more compressed by the capillary tension. The decrease in air content effected by the compacting process is, in part, a function of the work done in vibrating a unit volume of concrete and the surface area of the concrete exposed to the atmosphere. As is shown by Higginson[24] loss of air is greater from small specimens than from large for equal unit vibration, and loss of air from mass concrete in structures is relatively small. Also significant are the properties of the mixture, such as consistency, and the gradation, sand content, and particle shape of the aggregate, and the type of agent used to entrain the air.

Analysis of hardened concrete and mortar demonstrated consistently that compaction in laboratory specimens results in a material decrease in air content and also a decrease in the average size of air voids. However, spacing factor remains relatively constant. This is true whether the compaction is accomplished by hand rodding or tamping or a mechanical vibrator. Moreover, it is true regardless of whether the concrete or mortar contains an air-entraining agent. For example, three otherwise similar mixtures were pre-

TABLE 3—EFFECT OF HANDLING AND MOLDING ON THE VOID SYSTEM OF CONCRETE

Source of sample	Void parameters determined microscopically						
	Air content, percent	Chord intercept, in.	Specific surface, in.$^{-1}$	Voids intercepted per in.	Paste-air ratio	Paste content, percent*	Spacing factor, $\bar{L}$, in.
Natural gravel coarse aggregate, low air content							
Mixer	7.8	0.0135	296	5.775	3.04	23.74	0.0103
Unit weight can	3.4	0.0095	421	3.578	7.31	24.88	0.0130
Slump cone	2.9	0.0090	444	3.221	8.62	25.01	0.0135
Hand rodded bar	2.3	0.0071	563	3.238	10.94	25.16	0.0117
Vibrated cylinder	2.2	0.0065	615	3.384	11.45	25.19	0.0110
Natural gravel coarse aggregate, high air content							
Mixer	10.3	0.0065	656	16.884	2.23	22.92	0.0034
Unit weight can	7.8	0.0053	755	14.716	3.02	23.56	0.0040
Slump cone	7.1	0.0051	784	13.921	3.34	23.74	0.0043
Hand rodded bar	6.3	0.0046	870	13.694	3.80	23.94	0.0044
Vibrated cylinder	5.7	0.0039	1030	14.614	4.23	24.09	0.0041
Crushed limestone coarse aggregate, high air content							
Mixer	13.5	0.0057	702	23.658	1.70	22.94	0.0024
Unit weight can	8.5	0.0045	889	18.887	2.86	24.27	0.0032
Slump cone	8.4	0.0043	930	19.534	2.89	24.29	0.0031
Hand rodded bar	7.3	0.0040	1000	18.250	3.37	24.59	0.0034
Vibrated cylinder	6.5	0.0032	1250	20.312	3.82	24.80	0.0031

*Corrected for air content of the concrete.

pared with differing proportions of Agent J so as to effect a range of air content from about 3 to 7 percent. The mixtures were sampled immediately after mixing was complete, without molding or compaction. Other specimens of the concrete were obtained from the unit weight measure and slump cone with no further manipulation of the concrete than that required by the two tests. In addition, formed specimens were prepared by hand rodding and by vibration. After curing, the concrete was analyzed by the linear traverse method (Table 3). Air content and the average chord intercept were reduced by factors of nearly 2 or more, and α was increased proportionately by the manipulation incidental to molding and compaction of the specimens. However, $\bar{L}$ remained within narrow limits, ranging from 0.0103 to 0.0135 in. in the low air content series, and from 0.0034 to 0.0044 in. or from 0.0024 to 0.0034 in. in the two high air content series.

These results reaffirm previous conclusions that, within close limits, the magnitude of $\bar{L}$ is established by the inherent characteristics of the concrete mixture itself, including the nature and concentration of air-entraining agent. Handling, placing, and compacting operations do not significantly alter $\bar{L}$ but do materially decrease the air content and so minimize the loss of strength, (Fig. 3) which otherwise would accompany the development of a void system characterized by low values of $\bar{L}$. In the series illustrated in Fig. 3, a gain of

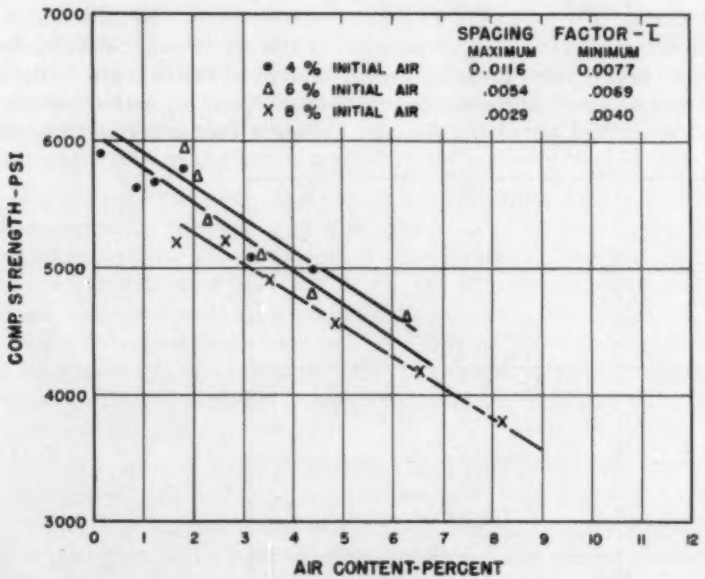

Fig. 3—Excellent correlation between strength and air content where specimens were vibrated different amounts. Note a gain of 300 psi for each percent air vibrated out, a considerable gain but not quite equal to strength lost by increasing air initially entrained

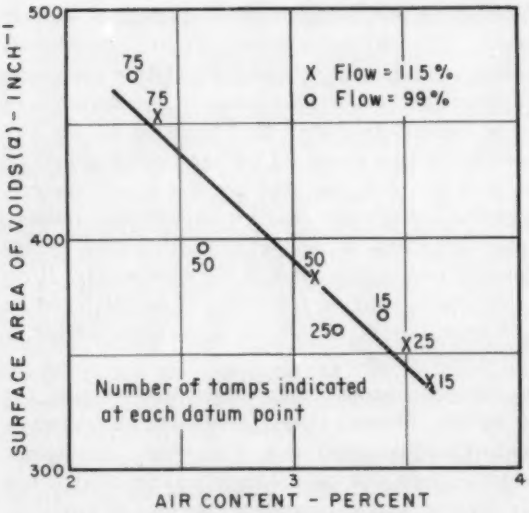

Fig. 4—Increase of surface area of voids α in mortar with increase of tamping

300 psi was obtained per percent decrease in air by vibration. A net loss of approximately 150 psi per percent of entrained air was obtained with increase of air content between the 6 and 8 percent concrete mixtures.

The change effected in the void size is controlled by the kind and duration of compaction employed in preparation of the specimens. With the number of tamps varying from 15 to 75, the air content of mortar bars containing no air-entraining agent decreased from 3.5 to 2.4 percent, and α increased from about 354 in.$^{-1}$ to 462 in.$^{-1}$ (Fig. 4). Concrete containing no air-entraining

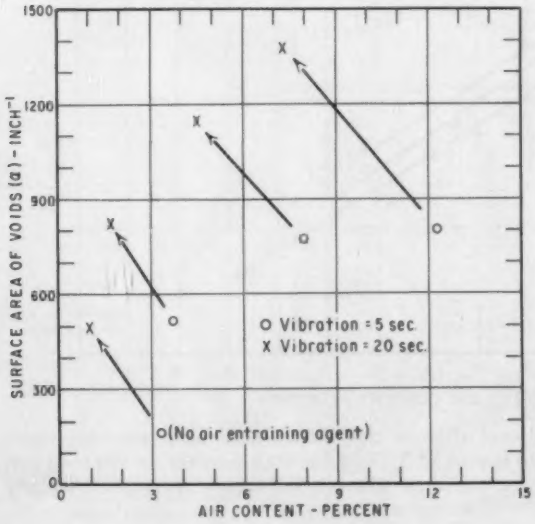

Fig. 5—Increase of surface area of voids α in concrete with increase in duration of vibration

TABLE 4—EFFECT OF VIBRATION ON THE VOID SYSTEM OF CONCRETE*

Void parameters determined microscopically†	Vibration time (seconds)	
	2	50
Air content, percent	6.70	1.20
Specific surface, in.$^{-1}$	870	1540
Number of voids intercepted per in.	14.572	4.614
Spacing factor, in.	0.0046	0.0058

*¾-in. maximum-size natural aggregate; 45 percent sand in aggregate; water-cement ratio = 0.51; 278 lb water per cu yd; slump = 2.9 in.; paste content = 26.8 percent; 587 ml Agent J per cu yd of concrete.
†See Table 5 for distribution of void sizes.

TABLE 5—EFFECT OF INTERNAL VIBRATION ON VOID SIZE DISTRIBUTION IN CONCRETE*

Diameter of voids, microns	Number of voids per cu cm of concrete†	
	2 sec vibration	50 sec vibration
20	155,839	68,869
40	96,180	29,887
60	19,830	15,641
80	14,666	2981
100	6014	1836
120	5847	838
140	1855	491
160	884	306
180	772	265
200	418	180
250	525	70.1
300	139	19.1
350	65.6	2.0
400	45.0	20.4
450	23.9	23.0
500	18.2	5.7
600	4.1	0.6
700	12.7	3.6
800	7.4	—
900	1.8	—
1000	13.9	—
2000	0.6	—
3000	0.7	—
4000	0.2	—
Total	303,163‡	121,438‡

*See Table 6 for mixture proportions.
†Calculated in accordance with the method established by Lord and Willis.[18]
‡Corresponding numbers of voids per cu in. of concrete are 4,968,841 and 1,990,368, respectively.

agent behaved in a like manner, the air content decreasing from 3.3 percent after 5 sec vibration to 1.0 percent after 20 sec vibration, and α increasing simultaneously from 160 in.$^{-1}$ to 496 in.$^{-1}$ (Fig. 5). Similarly, air content of concrete containing 12.3 percent of air after 5 sec vibration fell to 7.3 percent air during 20 sec of vibration, and α increased from 800 in.$^{-1}$ to 1380 in.$^{-1}$ (Fig. 5). The data were obtained on 3 x 6 x 1-in. slabs cut from the top and bottom section of 6 x 12-in. cylinders in which the entire concrete was vibrated for either 5 or 20 sec. A comparison of companion top and bottom slabs shows that vibration tends to produce a higher air content and lower value of α in the upper portion of the concrete than in the lower.

The characteristic increase in α indicates that the proportion of small voids increases with increase in compactive effort. This is demonstrated quantitatively by void section frequency distributions in concrete specimens from the same mix vibrated 5 sec and 30 sec (Fig. 6). By the additional vibration, the air content is reduced from approximately 8 percent to about 3 percent, and α is increased from 1000 in.$^{-1}$ to 1500 in.$^{-1}$. Void sections larger than 90 microns are reduced from 19.5 to 11.0 percent of the total number of voids, and void sections smaller than 50 microns are increased from 55.0 to 72.0 percent of the total number.

The effect of vibration upon the distribution of void sizes can be determined more quantitatively by the modified linear traverse method. This type of analysis was made on two concrete cylinders originally containing about 7 percent of air before being internally vibrated for 2 and 50 sec, respectively. Each analysis involved measurement of 1500 void sections. Increase in time of vibration from 2 to 50 sec decreased the air content from 6.70 to 1.20 percent and increased α from 870 to 1540 in.$^{-1}$ (Table 4). In spite of the great increase in α, the spacing factor, $\bar{L}$ increased only from 0.0046 to 0.0058 in.

The total number of voids per cu cm of concrete is indicated to decrease from about 303,000 to about 120,000, and the number of voids smaller than 40 microns decreases from about 250,000 to about 100,000. However, the proportion of voids smaller than 40 microns decreased only from 83 to 82 percent of the total number of voids in the concrete in each condition (Table 5). Voids larger than 60 microns, which constitute the greater proportion of the air content of the concrete vibrated only 2 sec, are greatly reduced in frequency by the additional vibration, probably as the result of movement to and escape from the upper surface of the specimen. However, voids in all size ranges are decreased in frequency.

The large voids are in part lost by escape at exposed surfaces and in part are disrupted by the process of compaction. The disruption occurs because distortion of a bubble, as by agitation, leads to such an increase in area of the air-water interface per unit volume of air that a net reduction in that area can be accomplished by splitting of the large bubble into two smaller spheres. For example, if a bubble is elongated by the shearing action incidental to vibration into a more or less cylindrical shape whose length exceeds about three times the diameter, surface tension will tend to pinch the bubble into two spherical bubbles with total surface area less than that of the distorted larger bubble. However, the film containing adsorbed air-entraining agent at the air-water interface will resist the distortion and consequent enlargement of the air-water interface because the momentary increase in area instantaneously increases surface tension in the film. This effect tends to resist rapid

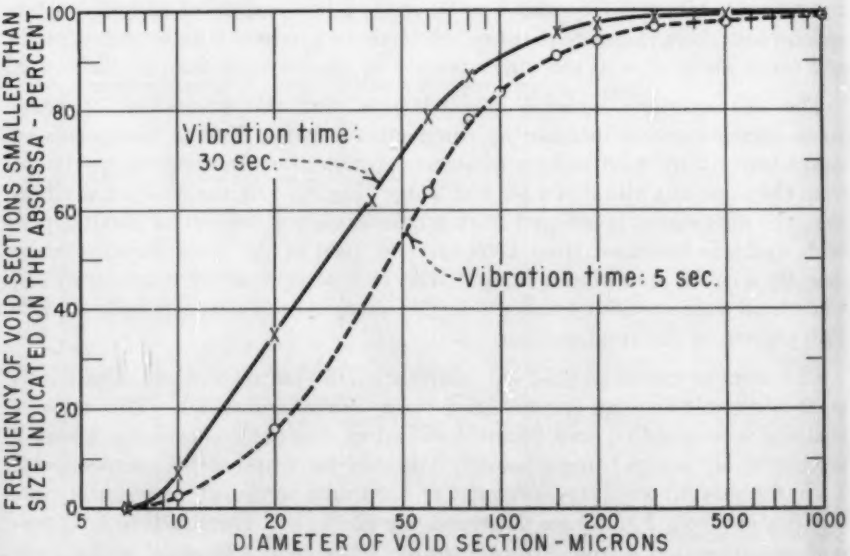

Fig. 6—Change of distribution of void section size with change of duration of vibration

TABLE 6—EFFECT OF VIBRATION TIME ON THE VOID SYSTEM OF CONCRETE

Specimen No.	Time of vibration, sec	Void parameters determined microscopically					Number of freeze-thaw cycles to 25 percent loss in weight
		Air content, percent	Chord intercept, in.	Specific surface, in.$^{-1}$	Voids intercepted, per in.	Spacing factor, $\bar{L}$, in.	
		3.0 percent air series*					
1	2	3.4	0.0066	603	5.12	0.0101	1050
2	6	2.6	0.0054	743	4.83	0.0090	1110
3	12	1.9	0.0056	720	3.42	0.0108	1160
4	20	1.7	0.0052	762	3.24	0.0104	620
5	30	1.4	0.0342	1170	4.10	0.0079	730
6	50	1.3	0.0397	1006	3.27	0.0092	700
		6.5 percent air series*					
7	2	6.7	0.0054	737	12.34	0.0056	1360
8	6	4.6	0.0048	840	9.66	0.0060	1630
9	12	2.7	0.0383	1043	7.04	0.0061	1330
10	20	2.4	0.0399	1001	6.00	0.0067	1380
11	30	1.9	0.0353	1133	5.38	0.0066	1340
12	50	1.2	0.0290	1380	9.14	0.0065	1010
		8.8 percent air series*					
13	2	8.2	0.0360	1112	22.80	0.0033	1540
14	6	6.8	0.0355	1125	19.12	0.0033	1780
15	12	5.0	0.0260	1541	19.26	0.0030	1910
16	20	4.6	0.0298	1340	15.41	0.0035	1690
17	30	3.4	0.0270	1479	12.57	0.0037	1440
18	50	2.9	0.0244	1639	11.88	0.0037	1830

NOTE: 1½-in. maximum size natural aggregate, water-cement ratio equals 0.50, 6 x 12-in. cylinders placed in two lifts, each lift vibrated for the time indicated. Tests conducted on 3 x 3 x 6-in. bars sawed from 6 x 12-in. cylinders.
*Air content determined from pressure method.

fluctuation of interface area. Also, the reduction of surface tension by the air-entraining agent will reduce the force tending to split the distorted larger bubbles. Hence, the process of disintegration of large bubbles into bubbles approximately half their size will be less in air-entrained mixes than in non-air-entrained mixes.

On the other hand, few very small voids are produced by disintegration of large voids during vibration. The distortion of bubbles during compaction will decrease with size of the bubbles, because difference in buoyancy and shearing stress at opposite sides of the bubbles will decrease progressively with size. Consequently, the process of splitting of distorted bubbles will be of little significance in small sizes (say below $d = 1$ mm). Of more significance in reducing comminution of large voids to very small ones is the great amount of work necessary to produce the enormous increase in surface area. For example, without an air-entraining agent approximately 400,000 ergs or 0.01 g-cal of work are required to reduce one bubble containing 1 cu cm of air to bubbles whose diameter is 8 microns, there being 2.74×10^9 of these. The work required is reduced proportionately as the surface tension of the water phase is reduced, and so use of an air-entraining agent increases the possibility that some small voids will be produced by this process in zones of maximum agitation in the plastic concrete.

Fig. 7 was prepared from data in Table 6 to demonstrate the influence of compaction on the air void parameters of three concretes and their interrelationship with resistance to freezing and thawing. The concretes represented contained 1½-in. maximum-size aggregate and were similar except for cement and air content. Specimens were vibrated for various periods ranging from 2 to 50 sec.

As shown previously, increasing periods of vibration resulted in decreased air content and increased α, but relatively unchanged spacing factor. Freezing and thawing resistance (Fig. 7), which in this instance is evaluated on the

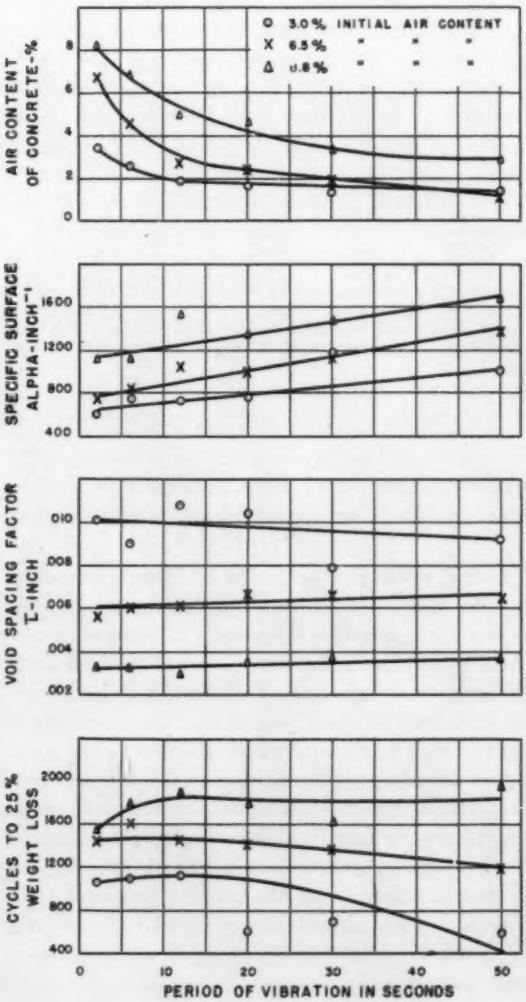
O 3.0% INITIAL AIR CONTENT
X 6.5% " " "
Δ 11.8% " " "
AIR CONTENT OF CONCRETE-%
SPECIFIC SURFACE ALPHA-INCH⁻¹
VOID SPACING FACTOR L-INCH
CYCLES TO 25% WEIGHT LOSS
PERIOD OF VIBRATION IN SECONDS

Fig. 7—Effect of vibration on parameters of void system and freezing and thawing resistance

basis of weight loss due to scaling and crumbling, did not follow the consistent patterns developed by the air void parameters of the three concretes, but increased in the high air, remained practically unchanged in the medium air, and decreased in the low air concretes with increased periods of vibration. Weight loss was used as the evaluation criterion because test specimens, 3 x 3 x 6-in. bars, were cut from the top and bottom halves of 6 x 12-in. cylinders and contained no inserts for length change measurements. Table 6 records average figures obtained from top and bottom specimens cut from the same 6 x 12-in. cylinder. On the average, top specimens were found to be 78, 74, and 78 percent as resistant to freezing and thawing as were companion bottom specimens from the low, medium, and high air concretes, respectively, although the spacing factor was affected but little. Compressive strength followed the same trend, top specimens from the three concretes developing 91, 82, and 84 percent of the strength of the bottom companions.

These data further strengthen the conclusion that at the time of discharge of the concrete from the mixer the air void parameters (including $\bar{L}$) of the concrete have been, to a great extent, determined. In Table 7 are recorded various air void parameters of specimens representing two or more of the three concretes shown in Table 6 which, according to microscopical analysis, contained, because of varying periods of vibration, approximately the same amount of air. In addition, the average $\bar{L}$ and freezing and thawing resistance of all specimens from each concrete are recorded for comparison with the same parameters of the individual specimens selected above. A study of this table indicates that a definite freezing and thawing resistance has also been established by the concrete, which remains relatively constant, despite variations in handling and vibration procedures, when the amount of air initially entrained equals or exceeds the so-called optimum for the particular water-cement ratio and maximum size of aggregate used. Note the close

TABLE 7—SELECTED AIR VOID DATA AND FREEZING AND THAWING RESISTANCE COMPARED

Specimen No.	Vibr., sec	Initial air content, percent	Final air content, percent	Specific surface, $in.^{-1}$	Spacing factor, $\bar{L}$, in.	Freezing and thawing cycles to 25 percent weight loss
2	6	3.0	2.6	743	0.0090	1100
Average 1–6					0.0096	900
9	12	6.5	2.7	1043	0.0061	1350
Average 7–12					0.0062	1340
18	50	8.8	2.0	1639	0.0037	1950
Average 13–18					0.0034	1700
1	2	3.0	3.4	603	0.0101	1050
Average 1–6					0.0096	900
17	30	8.8	3.4	1479	0.0037	1600
Average 13–18					0.0034	1700
3	12	3.0	1.9	720	0.0108	1150
Average 1–6					0.0096	900
11	30	6.5	1.9	1133	0.0066	1350
Average 7–12					0.0062	1340

agreement between values of $\bar{L}$ and cycles to 25 percent weight loss of individual mixes and average values of all mixes in a group.

When less than optimum air is entrained, freezing and thawing resistance is unaffected by a normal amount of vibration and handling but is detrimentally affected by overvibration. This apparent regression occurred even though values of $\bar{L}$ for these specimens were comparatively low, and indicates that total air content has in itself some influence apart from that of $\bar{L}$.

This confirms, in a single concrete, the trends developed in the series of three concretes under discussion in which the resistance to freezing and thawing of concretes having equal air contents (Table 7) was found to vary directly with the amount of air initially entrained.

EFFECTS OF MISCELLANEOUS FACTORS

Parts 2 and 3 of this series of papers deal primarily with the relationships which have been found to exist between the freezing and thawing resistance of 28-day moist cured air-entrained concrete and the void spacing factor, and between the void spacing factor and some of the most elementary properties of the fresh concrete and its placement. These properties, namely the type and proportion of air-entraining agent, the water-cement ratio, and compaction, have been developed from data adequately supporting the conclusions derived. Other factors, for which the data were not nearly so extensive, also appear to have a significant influence on the spacing factor and freezing-thawing resistance. These will be but briefly developed and no conclusions made.

Aggregate surface texture

The data plotted in Fig. 8 show the effect of surface roughness of sand particles on the bubble size and spacing as well as the effect on durability of concrete. Eleven aggregates, selected at random, were examined microscopically and divided into two groups, one representing sand particles with smooth surfaces, and the other representing rough surfaces. Each sand was processed to the same grading and incorporated into concrete at the same net water-cement ratio and about the same concentration of air-entraining agent. The effect of surface roughness of the sand particles on the bubble size and spacing is rather striking; the average value of α is 742 in.$^{-1}$ for the smooth sands and 1037 in.$^{-1}$ for the rough sands and the average values of the spacing factor are 0.0065 and 0.0045 in., respectively. The effect of the *angularity* of the sand particles could not be determined since all the sands were more or less angular in shape. None was well rounded.

A fairly good correlation of spacing factor and freezing and thawing resistance was shown with 7 of the 11 aggregates tested (Fig. 8). Four concretes expanded more than would be expected from the size of the spacing factor. A tenable explanation is that these four aggregates (No. 1, 2, and 3 from Trinity River, Calif., and No. 4 from Crescent Lake, Ore.), which had previously been rated inferior by petrographic examinations, contained appreciable

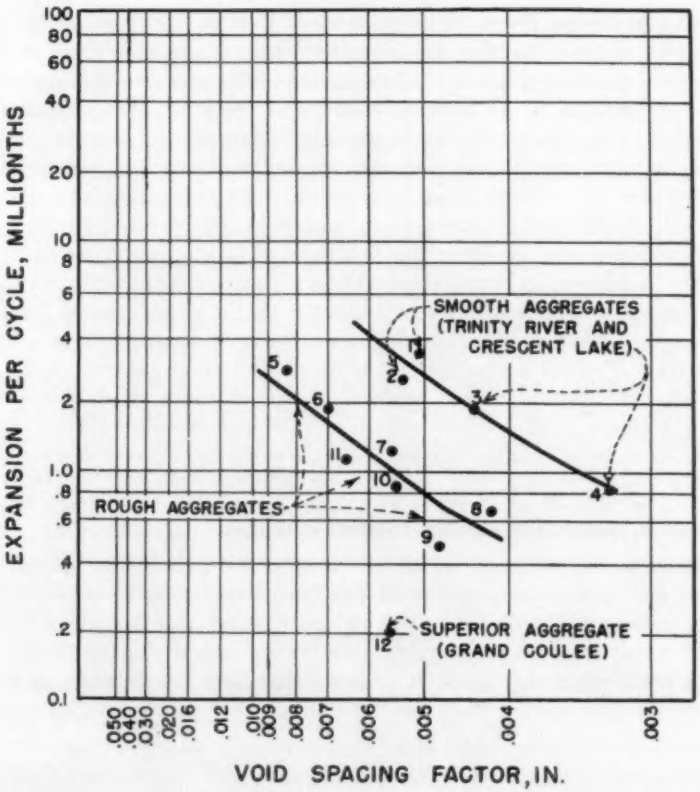

Fig. 8—With concretes containing different aggregates, the freezing and thawing expansion was influenced by the void spacing factor and by the quality of the aggregates. The void spacing factor was influenced by the surface texture of the sand particles

amounts of weathered and physically unsound materials. To round out the picture, a comparable concrete containing an exceptionally good aggregate (Grand Coulee) has been included in Fig. 8 in order to emphasize that even though a correlation was evident between the void spacing factor and the freezing and thawing resistance, there was still considerable spread at the same spacing factor, probably because of the effects of the thermal, bonding, porosity, and soundness characteristics of the aggregates.

Sand grading

A series of 12 mortar mixtures plus reruns of selected mixtures in which the fineness modulus of the sand was varied in small increments from 2.00 to 3.26 provided data for this section. The air-entraining agent requirement, bubble size, and spacing factor of those mixes for which air-void determina-

tions were made did not vary appreciably. Also, in the main series, neither the amount of air-entraining agent required for constant air content nor the freezing and thawing resistance of the moist cured concrete was significantly affected by changes in the fineness modulus of the sand. Investigations performed by H. L. Kennedy[25] on sand-water mixtures (no cement) indicated that the amount of air entrained was markedly affected by size grading of sand, the size No. 48-100 being most effective in entraining air. However, later work by Scripture, Hornibrook, and Bryant[26] revealed that in *cement* mortar and concrete mixtures the influence of sand grading on air entrainment was suppressed in proportion to the amount of cement added to the mix. Consequently, it appears that the influence of the sand grading on the air void system, although generally apparent, may be appreciable only in very lean mixes and, from a practical point of view, may be ignored in the richer mixes since a relatively large amount of cement fines, with adequate air, seems to insure a good void system. The influence of material finer than the No. 200 sieve on air content and size of air voids in concrete is discussed by Mather.[27]

Temperature, slump, and calcium chloride additions

From various unrelated series of tests investigating the simultaneous effects of slump and temperature, it has been found that a rise in temperature of concrete will reduce the total air content and also the size of air voids. The net result from these compensating effects was a spacing factor which was but little affected. Thus, it appears that with air content maintained constant, a better void system would be obtained during hot weather than in cold.

A decrease in slump, with water-cement ratio and amount of entrained air held constant, decreased the size of voids and spacing factor. Freezing and thawing data are not available from this series of mixtures; however, results from another series conducted at 70 F indicate reduced resistance with increased slump.

The addition of 1 percent calcium chloride to air-entrained concrete resulted in larger bubbles and a higher spacing factor. Since $CaCl_2$ is generally used in cold weather, the cumulative adverse effects on bubble size and spacing factor may be increased appreciably with possible serious effect on freezing and thawing resistance. Additional tests are planned on this phase.

CONCLUSIONS

1. The size distribution and frequency of air voids and concomitantly the spacing factor and freezing and thawing resistance are influenced by many factors, among the most important of which are water-cement ratio and the intensity of compaction. In general, but not always, an increase in freezing and thawing resistance accompanies any change which reduces the magnitude of the spacing factor.

2. Other factors remaining constant, the average size of voids and the computed spacing factor decrease with decreasing water-cement ratio.

3. Decrease of the rate of hardening of concrete will tend to increase the spacing factor.

4. Air content is progressively decreased and specific surface α of air voids is progressively increased by increasing periods of vibration, but little effect is noted on spacing factor of air-entrained concrete.

5. In concrete initially containing as much as or more air than recommended by the ACI Committee 613, the spacing factor is good to excellent and its magnitude is influenced little by vibration. For concrete containing less than the recommended amount of air, the spacing factor is relatively poor and more variable, and in general, freezing and thawing resistance is lower.

6. In a single concrete, variously vibrated, an optimum freezing and thawing resistance obtains at certain values of air content and spacing factor. The optimum spacing factor may or may not be the smallest in magnitude.

7. Specimens cut from the top half of 6 x 12-in. cylinders[22] consistently developed less resistance (approximately 77 percent) to freezing and thawing than did companion specimens cut from the bottom half of the same cylinders. Compressive strength followed the same trend, top specimens developing 86 percent of that developed by bottom specimens.

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Title No. 55-23

Stresses in Reinforced Concrete Sections Subject to Transient Temperature Gradients*

By HAROLD SAMELSON† and ABBA TOR‡

SYNOPSIS

Authors investigated stresses in walls of underground reinforced concrete cylindrical tanks containing liquids whose temperature varied from 50 to 500 F as a function of time. Stresses were checked for both the straight line temperature gradient, which represents a steady state of heat flow through the tank wall, and for the transient gradient. The transient gradient may be defined by a family of curves, each of which represents the temperature gradient at a given time station. Only the results for sections sufficiently removed from the ends where perturbational effects can be ignored are treated here.

Generally temperature stresses in structures are evaluated on the basis of a straight line temperature gradient only. This assumption may be justified in problems dealing with one dimensional heat transfer through thin structural material of relatively high conductivity. However, for relatively thick sections of low conductivity the transient gradient will produce a more severe stress condition. This stress condition which involves the entire section in a smooth variation may last for a considerable period and is not to be neglected under the assumption of a high localized state of stress which is relieved as plastic yield occurs.

The outlined solution is limited to problems of one dimensional heat flow.

INTRODUCTION

Notation

k = thermal conductivity of solid, Btu per hr per sq ft per deg F per ft

h = coefficient of heat transfer between surrounding medium at t_a and surface at t_o, Btu per hr per sq ft per deg F (equivalent to $\frac{k}{\Delta x}$ for boundary layer)

ρ = density of solid, lb per cu ft

c_p = specific heat of solid, Btu per lb per deg F

$t_n^{i\phi}$ = temperature, deg F, at position n, at time $i\phi$

t_a = ambient temperature, deg F

t_o = temperature at surface, deg F ($= t_n$ when $n = 0$)

t_b = original base temperature of solid, deg F

A = area, sq ft

Q = quantity of heat flowing through a layer, Btu

$\Delta\phi$ = time interval, hr

$\alpha = \frac{k}{\rho c_p}$ coefficient of thermal diffusivity, sq ft per hr

M = dimensionless constant $= \Delta x^2/\alpha\Delta\phi$

N = dimensionless constant $= \frac{h \Delta x}{k}$

Δx = thickness of slice, ft

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Linear heat conduction

The general expression for linear heat conduction through a layer of material of thickness Δx is

$$q = \frac{-\Delta Q}{\Delta \phi} = k A \left(\frac{-\Delta t}{\Delta x} \right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

For the steady state condition the amount of heat entering and leaving the layer Δx in a given time interval does not change (i.e. q is constant). For thick sections of low conductivity the steady state condition does not develop rapidly. The amount of heat entering any given layer during the time interval $\Delta \phi$ is not equal to the amount leaving and the difference of heat will cause a change of temperature in the layer with time.

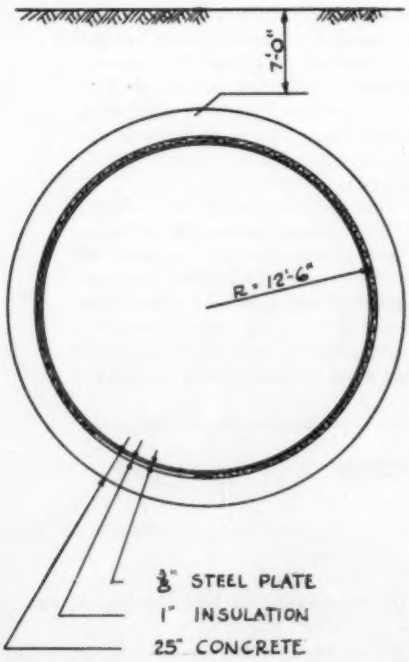


Fig. 1—Vertical cross section through tank

After sufficient time has elapsed, and provided that during the said period the heat conditions on the boundary remain constant, a steady state (straight line) gradient will be established. In general the time required for a steady state gradient to develop through a concrete wall is relatively long, and in many cases the actual temperature differential between the boundaries may not be reached in the concrete if the external heat conditions do not remain constant for a sufficient length of time. Where the source of external heat has been removed before the steady state gradient has been developed, the stresses caused by the transient gradients may be several times higher than those evaluated on the basis of the hypothetical steady state condition.

EVALUATION OF STRESSES IN TANK WALLS

A reinforced concrete cylindrical tank (Fig. 1) is subject to an internal temperature build-up from 50 to 220 F in a period of 30 days. The internal temperature remains constant (at 220 F) thereafter. The tank is covered by a layer of soil 7 ft deep. The temperature at the surface of this layer was assumed to be constant at 50 F. For the sake of simplicity it was assumed that the temperature of the soil at a radial distance of 7 ft from the tank wall

was also 50 F. The liquid is separated from the concrete wall by a ⅜-in. steel liner and a 1-in. layer of glass fiber type of insulation.

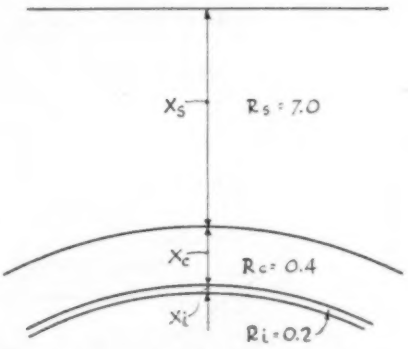


Fig. 2—Thermal resistances through concrete, insulation, and soil

Steady state gradient

The steady state gradient through the concrete wall is obtained by prorating the total temperature drop through the layers of insulation, concrete, and soil by the ratios of the thermal resistances R of the layers.

$$q = \text{constant} = \frac{\Delta t_i}{R_i} = \frac{\Delta t_c}{R_c} = \frac{\Delta t_s}{R_s} = \frac{\sum \Delta t}{\sum R} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$R = \frac{x}{k\,A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The values of R were obtained from Eq. (3) (Fig. 2).

	insulation	concrete	soil
Δx (thickness, ft)	0.083	2.08	7.0
k (conductivity in Btu per hr per sq ft per deg F per ft)	0.035	0.44	0.083
A (area, sq ft)	1.0	1.0	1.0*
R (thermal resistance)	2.36	4.71	84.0 ($\Sigma R = 91.1$)

From Eq. (2), $\sum \Delta t / \sum R = (220 - 50)/91.1 = 1.87$ Btu per hr, and Δt for concrete $= 1.87 \times 4.71 = 8.8$ F. This is the steady state gradient through the concrete wall.

Transient gradient

The transient state gradients were obtained† for increments of 5 days (Fig. 3) by evaluating the temperatures of concentric slices of the concrete wall at the end of time intervals $\Delta\phi = 5$ days. The successive 5-day transient gradient curves approach the steady state (straight line) gradient as shown in the figure.

In the problem under investigation the critical transient gradient G_{tc} occurs approximately 30 days after the beginning of the temperature build-up.

*The variation of A due to the curvature of the wall was neglected. In the example above its extreme value (between the wall surface and the midthickness) does not exceed 8 percent.

†The numerical method used to establish these gradients can be found in *Heat Transmission* by William H. McAdams, published by McGraw-Hill Book Co., New York, 1933: Chapter 3, Article II, and Chapter 7, Articles I and IV. The computations and the outline of the method are given in the appendix to this paper, p. 383.

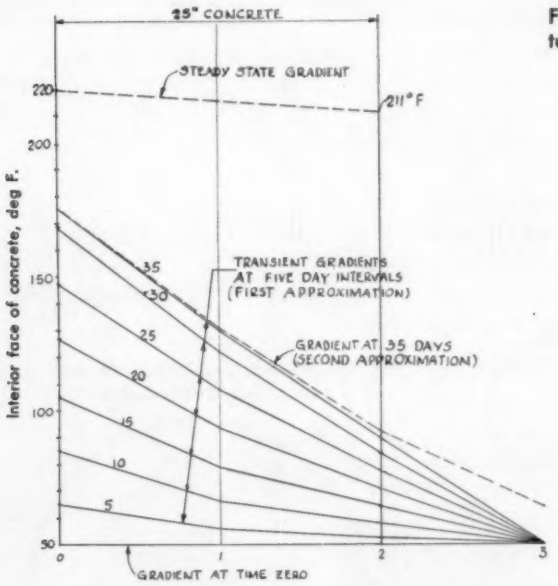


Fig. 3—Transient temperature gradients through concrete walls

It is represented by a curve approximated by means of two straight lines representing partial gradients between the inner surface of the wall and its middle surface, and between the middle and the outer surfaces. The partial gradients consist of a gradient of 46.3 F through the interior 12.5 in. of the wall and a gradient of 38 F through the exterior 12.5 in.

Note that in spite of the fact that the interior temperature of the tank has reached the final value of 220 F, the face of concrete close to the source of heat at the same time has reached a temperature of only 168 F. The total critical gradient $G_{tc} = 46.3 + 38.0 = 84.3$ F as against 8.8 F for the steady state gradient which occurs after the interior face of the concrete has reached the maximum temperature of 220 F.

Calculating the stresses

Stresses were determined by the use of the following conditions:

1. Temperature gradient is symmetrical and constant for all sections at a given time station.
2. External load is zero.

These conditions require that the load acting on any section be constant. The shear at any section must be zero to satisfy symmetry. The thrust at any section must be zero in order to satisfy the conditions of equilibrium, namely that the radial components of the thrust acting on any circumferential element must equal the resultant of external loads on the element in the radial direction. Since the latter is zero from Condition 2 above, the thrust must also be zero.

Therefore the constant load acting at every point must be a couple $Ca = Ta$, when C is the total compression and T the total tension acting on the section and preventing the development of tangential strains. Thus the temperature gradient will cause compressive stresses in the inner portion of the wall (close to the source of heat) and tensile stresses in the outer portion.

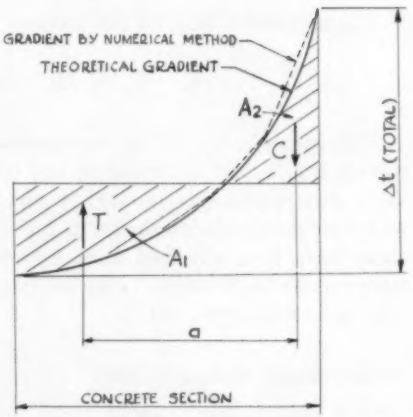


Fig. 4—Total compression and tension forces acting on the section

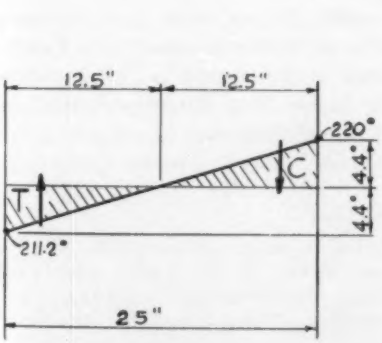
In order to obtain the values of $C = T$, a line perpendicular to the wall surfaces has to be found such that the areas contained between it and the temperature gradient curve be equal; $A_1 = A_2$ (Fig. 4). The areas thus obtained represent the couple $C = T$ multiplied by a coefficient of proportionality.

The balanced stress of the extreme fiber in tension or compression is proportional to the prevented free-body deformation of the section (Eq. 4):

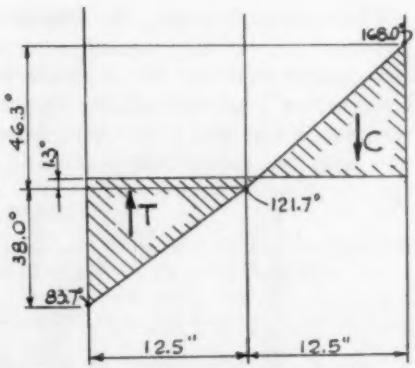
$$f = E \propto \Delta t \quad \ldots\ldots (4)$$

where E = modulus of elasticity of concrete (assumed to be 3×10^6 psi), and $\propto$ = coefficient of linear expansion of concrete (assumed as 5.5×10^{-6}).

$$C = T = A\ E \propto \quad \ldots\ldots (5)$$



Steady state gradient



Transient temperature gradient at ϕ = 30 days

Fig. 5—Stress distribution through section

The moment acting on the section is

$$M_t = A\,E \propto a \qquad (6)$$

This moment has to be superposed on any other moments acting on the section due to liquid pressures, soil pressures, and other loads. The approximate distributions of stresses in the tank wall under the steady state and transient gradients are shown in Fig. 5. The values for the transient gradient could have been obtained more accurately if the number of slices Δx through the concrete were larger. The moments across the section due to these gradients are by Eq. (6):

for the steady state gradient

$$M_{t_s} = \frac{(4.4 \times 12.5 \times \tfrac{1}{2})\,(5.5 \times 10^{-6})\,(3 \times 10^{6})\,(2/3 \times 25)}{10^{3}} = 7.56 \text{ ft-kips per ft}$$

for the critical transient temperature gradient

$$M_{Gt_c} = \frac{(45 \times 12.5 \times \tfrac{1}{2})\,(5.5 \times 10^{-6})\,(3 \times 10^{6})\,(2/3 \times 25)}{10^{3}} = 75.24 \text{ ft-kips per ft}$$

The moment across the wall section due to the transient gradient is almost ten times as large as the one caused by the steady state gradient, and thus the induced stresses will be considerably higher for the transient condition.

As the transient gradient curves in the vicinity of the critical gradient do not change shape or direction abruptly (Fig. 3), it is safe to assume that the high stress condition will exist for a period measured in days or weeks.

The relation between the internal moments (or extreme fiber stresses) and time elapsed since beginning of temperature build-up is shown in Fig. 6. It is readily seen that the moments produced in the section by the transient temperature gradients will be considerably higher than those evaluated on the basis of the final steady state temperature differential.

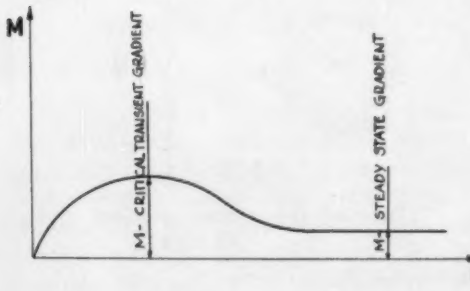


Fig. 6—Curve of moment versus time

Rapid rise of internal temperature

Consider the tank of Fig. 1 subject to a rapid rise of internal temperature, 220 to 500 F in a period of 8 hr, following which it drops instantaneously back to its initial value of 220 F.

The concrete wall has been subdivided into six equal concentric slices and the transient gradient at the end of 8 hr has been evaluated according to McAdams (Fig. 7). The interior face of the concrete wall will reach 273 F only after 8 hr, following which it will begin to cool off. The total maximum temperature gradient will be $G_{tc} = 273 - 211 = 62$ F. If the steady state condition were assumed between 500 and 220 F, (disregarding the fact that this gradient would not have sufficient time to develop under the conditions of the problem) the gradient through the concrete would be (500 F − 220 F) 0.38/0.58 = 183 F. In this case the comparison of moments caused by the steady and transient gradients would be: $M_{ts} = 157$ ft-kips per ft for the steady state, and $M_{Gt_c} = 33$ ft-kips per ft for the transient gradient after 8 hr.

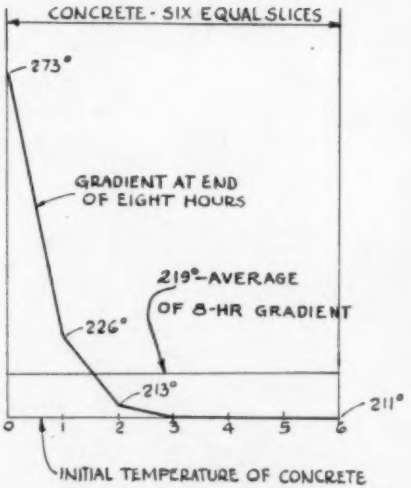


Fig. 7—Transient gradient at $\phi = 8$ hr

CONCLUSION

By neglecting the effect of time on the magnitude of temperature gradients for different heat boundary conditions, and assuming steady state gradients only, the moments produced in reinforced concrete structures by temperature changes of relatively long duration are consistently underestimated. On the other hand, in the case of abrupt temperature changes, the use of the steady state gradient will yield higher values of moments. To get a closer approximation of moments and stresses due to temperature changes in relatively thick sections of low conductivity material the transient gradients should be used whenever temperature loads are a substantial part of the total load.

ACKNOWLEDGMENTS

This paper is based on a report prepared by Amman and Whitney, Consulting Engineers, for the General Electric Co., Hanford Atomic Product Operation, Chemical Processing Department under Consulting Agreement No. CA 147. H. W. Stivers was in charge for the General Electric Co. Edward Cohen was project engineer for Ammann and Whitney.

APPENDIX

Determination of transient gradients through the wall section

The section under investigation is divided into equal slices (Fig. 8a). At any time a heat balance equation for any interior slice may be written, which states that the difference be-

tween heat entering the slice and that leaving the slice during a time interval changes the temperature of the layer during that interval. This is expressed according to McAdams as follows:

$$\frac{k\,A\,(t_{n-1}{}^{i\phi} - t_n{}^{i\phi})}{\Delta x} - \frac{k\,A(t_n{}^{i\phi} - t_{n+1}{}^{i\phi})}{\Delta x} = \frac{A\,\Delta x\,c_p\,\rho(t_n{}^{(i+1)\phi} - t_n{}^{i\phi})}{\phi} \quad \ldots\ldots(7)$$

(heat entering layer) − (heat leaving layer) = residual heat

Substituting $k/\rho\,c_p = \alpha$ and $\Delta x^2/\alpha\,\phi = M$, and solving for $t_n{}^{(i+1)\phi}$ yields

$$t_n{}^{(i+1)\phi} = \frac{t_{n-1}{}^{i\phi} + (M-2)\,t_n{}^{i\phi} + t_{n+1}{}^{i\phi}}{M} \quad \ldots\ldots(8)$$

For the values $n = 1$, $i\,\phi = 0$, and $t_1 = t_1{}^{i\phi}$, Eq. (8) becomes

$$t_1{}' = \frac{t_o + (M-2)\,t_1 + t_2}{M} \quad \ldots\ldots(9)$$

$t_1{}'$ is the temperature at Station 1 after the first time interval. The heat balance for a concrete layer at the surface is as follows (see Fig. 8b):

$$hA\,(t_a{}^{i\phi} - t_o{}^{i\phi}) - \frac{kA\,(t_o{}^{i\phi} - t_1{}^{i\phi})}{\Delta x} = \frac{A\,\Delta x\,\rho c_p(t_{0.25}{}^{(i+1)\phi} - t_{0.25}{}^{i\phi})}{2\phi} \quad \ldots\ldots(10)$$

assuming that $t_o{}^{(i+1)\phi} - t_o{}^{i\phi} = t_{0.25}{}^{(i+1)\phi} - t_{0.25}{}^{i\phi}$. Substituting $(h\,\Delta x)/k = N$, and $(\Delta x^2)/\alpha\,\phi = M$,

$$t_o{}^{(i+1)\phi} = \frac{2\,N\,t_a{}^{i\phi} + (M - 2\,N - 2)t_o{}^{i\phi} + 2\,t_1{}^{i\phi}}{N} \quad \ldots\ldots(11)$$

For the values $i\phi = 0$ and $t_o{}' = t_o{}^{1\phi}$ the equation becomes

$$t_o{}' = \frac{2Nt_a + (M - 2N - 2)\,t_o + 2t_1}{M} \quad \ldots\ldots(12)$$

$t_o{}'$ is the temperature at the surface after the first time interval.

The above equations may be used to determine the temperature gradients at any time interval by using the following numerical procedure. The number of slices is selected, which fixes Δx

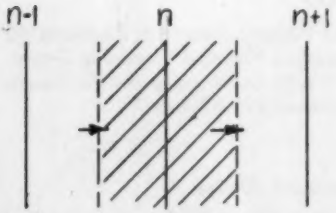


Fig. 8a—Interior slice

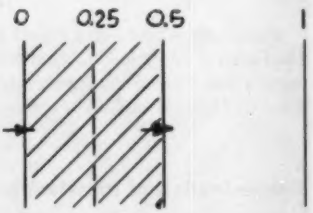


Fig. 8b—Surface slice

TABLE 1—SOLUTION OF EQ. (9) AND (12)

n, (time stations in intervals $\Delta\phi$)	$t_o^{n+1} = 0.4\ (t_a^n + 0.5\ t_o^n + t_1^n)$				$t_1^{n+1} = 0.2\ (t_a^n + 3t_1^n + t_2^n)$				$t_2^{n+1} = 0.2\ (t_1^n + 3t_2^n + t_3^n)$			
	Surface temperature at time station				Section 1 temperature at time station				Section 2 temperature at time station			
	t_a^n (avg)	$0.5t_o^n$	t_1^n	t_o^{n+1}	t_o^n	$3t_1^n$	t_2^n	t_1^{n+1}	t_1^n	$3t_2^n$	t_3^n	t_2^{n+1}
(−1)				(50.0)				(50)				(50)
0	51.69	25	50	50.7	50	150	50	50	50	150	50	50
1	55.07	25.35	50	52.2	50.7	150	50	50.14	50	150	50	50
2	58.45	26.1	50.14	53.9	52.21	50.4	50	50.52	50.14	150	50	50
.												
.												
8.4	(corresponds to 5 days)											
	78.73	.....		64.9	.....	.....	.....	55.3	.....		.....	51.4
50	(corresponds to 30 days)											
	220.0	.....		168.0	.....			121.7	.....		.....	83.7

and therefore N. M is selected, preferably as an integer. $\Delta\phi$ is fixed by a selection of M. (Nonintegral values of M may be required where it is desired to specify $\Delta\phi$, Δx, and α, or for a heterogeneous section where α and Δx do not permit the use of a single value of M.)

Having selected M and N, and knowing the thermal boundary conditions, Eq. (12) can be used to determine the temperature of the "zero station" at the end of the first time interval t_o'.

The temperature of any interior slice at the end of the first time interval t_n' may be obtained from Eq. (9). These values of t_n' can be inserted in turn into Eq. (9) and (12) and values of t_n'' (at the end of the second time interval) obtained. This procedure may be repeated for as many time intervals as required, depending on the total duration of the "thermal loading."

The above solution yields a family of curves defining the gradients through the section at different time intervals (Fig. 3).

The limiting value of the temperature gradient is a steady state condition for which each face of the section is at the temperature of the surrounding medium. This gradient is a straight line for a homogeneous material.

Evaluation of transient gradients for gradual build-up of temperature

Physical constants:

Concrete: $\rho = 150$ lb per cu ft
$c_p = 0.20$ Btu per lb per deg F
$k = 0.50$ Btu per hr per sq ft per deg F per ft

Insulation: $h = \dfrac{k}{\Delta x} = 0.42$ (equivalent to 1 in. of insulation).

Let $N = 1$ and $M - 2N - 2 = 1$ (or $M = 5$); hence $\Delta\phi = 14.3$ hr. One 5-day interval (for which gradients were found) includes $(24 \times 5)/14.3 = 8.4$ time intervals.

Assuming a straight line build-up in the fluid (from 50 F to 220 F) t_a (average) was taken as average temperature of fluid between time stations n and $n + 1$. Using the chosen values of M and N, Eq. (9) becomes for t_1' at any interior slice:

$$t_1' = \frac{t_o + 3t_1 + t_2}{5} = 0.2\ (t_o + 3t_1 + t_2)$$

and Eq. (12) becomes for the surface

$$t_o' = 0.4\ (t_a + 0.5t_o + t_1)$$

Using Eq. (9) and (12) in this simplified form and the average temperature of the fluid for a given time interval as the temperature of surroundings, the values of t_o, t_1, and t_2 have been

evaluated and tabulated (Table 1). These results have been plotted (Fig. 3) and used as a basis for the evaluation of moments (Fig. 5b).

It should be noted that for the purpose of this evaluation the temperature at Station 3 (Δx = 12.5 in. away from the exterior face of the concrete) was kept constant. Thus the value of t_2^n in Table 1 was taken as 50 F for all values of n. The error due to this assumption is small for a period of 30 days, as can be seen by comparing the original and corrected lines in Fig. 3 (second approximation).

This error can be reduced by allowing the temperature at Station 3 to vary and assuming it to be constant at Station 4, farther away from the outer face of the concrete section. A similar analysis has been run between Stations 2, 3, and 4, using the values t_2^n obtained from the table. This yields new values for t_2^{n+1}, greater than 50 F, which in turn would be introduced to correct values of t_2. The same procedure could be repeated to obtain corrected values of t_1. This correction was evaluated for ϕ = 35 days and was found to be negligible. The corrected gradient at time station ϕ = 35 days is shown (in dotted line) on Fig. 3.

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Title No. 55-24

Shear Strength of Lightweight Reinforced Concrete Beams*

By J. A. HANSON†

SYNOPSIS

This report describes tests employed and the results obtained in a study of the resistance of lightweight structural quality concrete in diagonal tension. A comparison of the shear strengths of the various beams, studied on an equal compressive strength basis, showed somewhat better performance of the sand-and-gravel concrete. However, when the comparison was extended to include a range of shear strength data reported in the literature by other investigators, the particular lightweight aggregate concretes considered here were as strong in shear, on an average basis, as the normal weight concretes.

Most of the beams sustained an ultimate load higher than that which caused diagonal cracking, but analysis of test data showed that such excess strength was related to location of the diagonal tension crack., This location was, within limits, a matter of chance, and load capacity above diagonal cracking load was therefore not a dependable quantity. This conclusion is believed generally applicable to beams without web reinforcement in which true diagonal tension may cause or initiate failure.

Deflections of the lightweight concrete beams were 15 to 35 percent greater than those of normal weight beams of equal strength.

INTRODUCTION

Experimental and analytical investigations of reinforced concrete over the past 60 years have been directed principally to studies of the flexural performance of concrete structural members. Only a few investigations have been concerned with diagonal tensile strength. Laupa, Siess, and Newmark[1] have given a summary and bibliography of the results of shear investigations of sand-and-gravel concretes. Richart and Jensen[2] studied the shear strength of beams cast from Haydite (an expanded shale) aggregate concrete. Reported results of the several investigations of shear and diagonal tension include a wide range of testing procedures, concrete quality, and test specimen design.

The introduction of many new lightweight aggregates and the growing scope of structural applications of lightweight aggregate concrete have increased the need for more information on the shear properties of these concretes. This report deals with seven of the considerable number of com-

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†Member American Concrete Institute, Development Engineer, Products and Applications Development Section, Portland Cement Association, Chicago, Ill.

TABLE 1—MIX PROPORTIONS, STRENGTH, AND ELASTIC PROPERTIES OF CONCRETES FOR SHEAR BEAMS

Aggregate No.	Fine aggregate, percent	Quantities per cu yd of concrete				Percent air (rollometer)	Unit weight of unhardened concrete, lb per cu ft	Gross W/C by weight	Slump, in.	Compressive strength, psi, 28 days		E_c,† 10^6 psi, 28 days		Poisson's ratio, 28 days	
		Water, lb	Cement		Aggregate					Wet	Dry	Wet	Dry	Wet	Dry
			sacks	lb	Fine, lb	Coarse, lb									
3000-psi Series															
2	45	438	6.44	606	708	716	10.2	91.4	0.72	1½	3470	3680	1.69	1.60	0.20	0.20
3	45	337	5.13	483	948	744	12.5	93.0	0.70	2	3090	3310	1.94	1.89	0.19	0.20
4	55	513	4.26	401	1051	734	9.0	100.0	1.28	1¾	2700	2980	1.52	1.60	0.17	0.18
5	55	427	5.97	562	1095	604	9.7	99.5	0.76	2½	3340	3490	2.07	2.00	0.22	0.19
6	55	439	7.20	677	1189	651	5.5	109.5	0.65	1¾	3650	3670	2.76	2.44	0.21	0.21
7	55	464	6.30	593	1197	696	8.8	109.3	0.78	1¾	3070	3210	2.33	2.19	0.18	0.17
8	48	228	3.89	366	1595	1733	5.7	145.3	0.62*	1¾	3750	4020	3.22	3.82	0.17	0.18
4500-psi Series															
2	40	443	8.50	800	623	771	6.0	97.7	0.55	1	5330	5350	2.09	1.81	0.20	0.20
3	40	357	6.76	636	835	805	6.5	97.5	0.56	2½	3960	4060	2.11	1.98	0.19	0.17
4	50	479	5.76	542	938	792	8.9	101.9	0.88	2¾	4550	4890	2.03	1.87	0.19	0.19
5	50	420	7.83	737	993	662	8.0	104.1	0.57	2¼	4740	4790	2.34	2.23	0.19	0.22
6	50	412	8.89	836	1042	694	4.5	110.5	0.49	1¼	4880	4870	3.08	2.77	0.24	0.23
7	55	454	7.07	664	1169	680	7.2	110.0	0.68	1½	4170	4240	2.49	2.33	0.20	0.18
7	50	442	7.78	732	1084	708	5.0	112.1	0.60	1¾	4950	5200	2.85	2.63	0.18	0.18
8	39	223	4.88	459	1281	2002	5.8	146.8	0.49*	2	5100	5380	3.82	4.01	0.20	0.18
7000-psi Series															
9	30	327	10.23	962	646	923	4.0	105.9	0.34	¼	6610	6910	3.11	2.92	0.22	0.20
4	35	459	6.84	643	711	1109	4.0	108.2	0.71	½	6840	7000	2.06	2.03	0.17	0.18
8	25	227	5.94	559	856	2567	1.1	155.8	0.41*	½	7630	8410	4.03	4.58	0.21	0.19
9000-psi Series															
4	35	474	10.22	961	632	989	3.6	113.2	0.49	¾	8140	8160	2.42	2.24	0.19	0.18
8	25	271	10.12	952	742	2228	1.7	155.3	0.29*	½	9610	10680	4.75	5.07	0.23	0.21

*Net water-cement ratio by weight.
†Secant modulus of elasticity at 0.3f'_c.

mercially available lightweight aggregates. To accommodate this group of aggregates within reasonable bounds of laboratory effort it was found necessary to limit the study to one size of test beam and one loading condition. The variation which has been found between lightweight aggregates, even of similar types, indicates the need for individual manufacturers of lightweight aggregates to develop their own design data through sound investigations of their respective products.

This study is part of an investigation in progress at the Portland Cement Association Laboratories to augment available technical information necessary for practical and economical design of structural concrete using lightweight aggregate. Shideler[3] has reported data from this program on compressive and flexural strength, modulus of elasticity, bond, creep, and drying shrinkage of eight lightweight aggregates and one normal weight aggregate. In the present shear study the same aggregates were used with one exception, and also the same concrete strengths, 3000, 4500, 7000, and 9000 psi. Aggregate 1 of the first report has been replaced by Aggregate 9 which is manufactured by the same process. Sixteen beams were cast using the 3000 and 4500-psi concrete, and five beams were fabricated from 7000 and 9000-psi mixes. These high strength concretes are representative of those used in some precasting or prestressing operations. Comparison of the performance of the lightweight beams and of sand-and-gravel concrete beams is made on the basis of equal compressive strengths.

TESTING PROCEDURE

Moody, Viest, Elstner, and Hognestad[4] have shown that the loads required to produce diagonal tension failure in a beam depend on cross section, concrete strength, amount of longitudinal steel, amount of web reinforcement, nature of the loading, and span. In addition to these six variables, this study of lightweight concrete brings in various aggregates. Since only a limited investigation of shear strength could be accomplished, no attempt has been made to cover all such factors, but the principal variables affecting shear were limited to compressive strength and type of aggregate. The performance of lightweight concrete beams as compared to normal weight beams in this report is then qualified by the particular dimensions and loading of the chosen test specimen. The probability of flexural failure occurring before or together with shear failure increases with increasing span-depth ratio and also increases with increasing distribution of the total load. In order that shear, rather than flexure, should be a governing factor, the test specimen chosen was a 6 x 12-in. beam with a 6½-ft span loaded at the third points. Although it was desired to eliminate steel percentage as a variable, this could not be done completely because of the wide range of concrete strengths involved. As a compromise, 2.5 percent steel was selected for the lower strength concretes and 5 percent for the higher. No web steel was used.

Aggregates and concrete mixes

The aggregate identification numbers used in the shear investigation were the same as those employed for the plain concrete tests. Complete descriptions including grading and physical properties of the various aggregates are given by Shideler,[3] and an abbreviated description is given below:

Aggregate 2 is an expanded shale produced in a rotary kiln. Material passing No. 4 sieve is obtained by crushing.

Aggregate 3 is an expanded shale produced in a rotary kiln. All particles are rounded and have a smooth shell.

Aggregate 4 is an expanded clay produced in a rotary kiln. Most of the material is crushed but is not harshly angular.

Aggregate 5 is an expanded slate produced in a rotary kiln. The coarser particles are somewhat angular and porous. Material passing No. 4 sieve is obtained by crushing.

Aggregate 6 is an expanded blast furnace slag produced by spraying a controlled amount of water on a thin layer of molten slag. All particles are angular and porous and most of the finer sizes are obtained by crushing.

Aggregate 7 is produced by burning on a sintering grate a carbonaceous shale from anthracite coal processing. All sizes are obtained by crushing and particles are very sharp and porous.

Aggregate 8 is Elgin sand and gravel. The gravel is well rounded.

Aggregate 9 is an expanded shale made in a rotary kiln. Particles are rounded and have a smooth shell.

The concrete proportioning and placing procedure for the shear beams were also similar to those employed in the plain concrete tests; details may be found in Reference 3. Dry aggregates were used for all lightweight concretes and for the lower strength sand-and-gravel concretes. Aggregate for higher strength sand-and-gravel concrete was soaked in water 24 hr prior to mixing. For comparison purposes the final mix quantities, unit weights, compressive strengths, and static moduli of elasticity of the shear beam concretes are given in Table 1.

Reinforcing steel

Two samples of the intermediate grade reinforcing steel used were tested for yield point and modulus of elasticity with the following results:

	Yield point, psi	Modulus of elasticity, psi
Sample 1	48,500	27,800,000
Sample 2	48,100	27,300,000

Theoretical considerations of beams

In accordance with the plan followed in the plain concrete tests, shear strength of beams was investigated for four nominal concrete compressive strengths: 3000, 4500, 7000, and 9000 psi. To avoid flexural failures, steel areas were greater than those generally encountered in service structures. Preliminary analysis by straight-line theory indicated that a 6 x 12-in. normal weight beam without web reinforcement or compression steel, loaded at the third points of a 6 ft 6 in. span, would fail by diagonal tension when the maximum concrete compressive stress approached 0.45 f_c' and steel stress approached 20,000 psi. The indicated steel requirements for this condition were 2.1 percent for concrete of 4500-psi compressive strength and 4.8 percent for 9000-psi concrete. Accordingly, it was decided to use two #8 bars (2.5 percent) for beams of lower strength concretes and four #8 bars (5.0 percent) for beams of higher strength concretes. All steel was intermediate grade, deformed in accordance with ASTM A 305.

As a final check on the beam cross section and steel percentage, investigations were made using concrete properties (Table 2) taken from averages of the results of the plain concrete tests and assuming the modulus of elasticity of the reinforcing steel as 29×10^6 psi.

The analysis showed that for concrete stresses of 0.45 f_c', the bond and steel stresses in beams of both normal weight and lightweight concrete were within the limits of the 1956 ACI Building Code, except for the lightweight beams in the 4500-psi and 9000-psi concrete strength groups. In these exceptions, the steel stresses were computed at 23,200 and 26,300 psi, respectively. For the same concrete stresses, estimated values of nominal shear were 40 to 150 percent above the allowable limit for the lower strength concretes and 270 and 500 percent above the allowable limit for the higher strengths.

Fabrication and curing of specimens

All beams were cast in plywood forms. The reinforcing bars were supported from the bottom form on steel chairs at a height to give an effective depth of 10½ in. The four #8 bars necessary for the beams of higher strength concretes were bundled[5] horizontally in pairs to provide clearance for the larger size aggregate during concreting.

All batching was by weight. The concrete was mixed in a 1¾-cu ft pan-type mixer. The aggregates and about two-thirds of the mixing water and air-entraining agent were mixed 2 min; then the cement and remaining water and agent were added and mixing was continued for an additional 3 min. A blend of equal parts of four commercial Type I cements was used. The concrete was placed in the beam mold in three 4-in. lifts. The concrete volume of one lift plus two 6 x 12-in. cylinders constituted one batch.

Each layer of concrete was vibrated after the full batch was placed. For the 3000 and 4500-psi mixes an internal vibrator was used. A table vibrator of special construction was used to compact the no-slump higher strength concretes. This table was powered by an electromagnetic unit that transmitted 3600 vibrations per min. The plywood form was clamped to the table top and then each of the three lifts was vibrated for several minutes. Segregation of aggregate was not found and no lift stratification was noted in the hardened concrete. The cylinders were consolidated by vibration in the same manner as the beams.

All beams and cylinders were covered with wet burlap for the first 24 hr after casting, except those of the high strength sand-and-gravel concretes (7000 psi and above) which were "ponded" for the 24-hr period. Ponding entails covering with water that portion of the concrete exposed in the form. At the end of 1 day, the forms and molds were stripped and the specimens were placed in a moist room until age 7 days. At this time, all beams and half of the 6 x 12-in. cylinders were stored in laboratory space at approximately 75 F and 30 percent relative humidity until tested. All tests were performed at 28 days.

Instrumentation

Fig. 1 shows the loading equipment details and location of SR-4 strain gages used on the beams. Steel strains were measured near the probable location of the diagonal tension crack and also in the center of the pure flexure section. The gages were A-1 type and required no waterproofing since they were cemented to the exposed steel in recesses in the concrete after moist curing. The recesses for the gages near the ends of the beam were located on one side and for the center on the opposite side. Maximum compressive strains in the concrete were measured at the load points and at the center of the span. All gages for measuring concrete

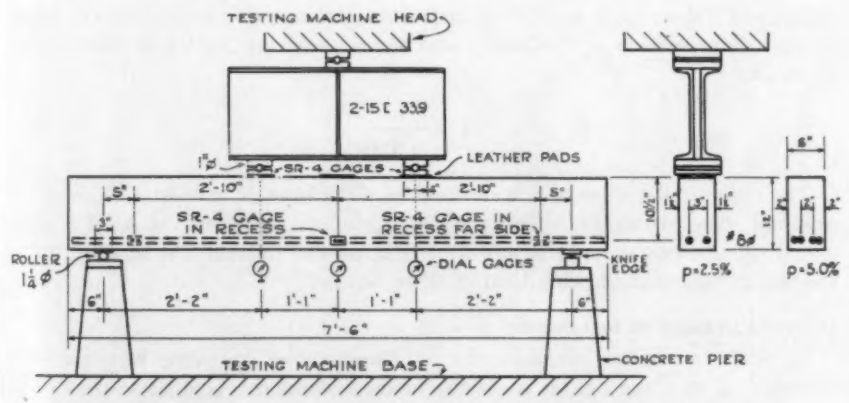

Fig. 1—Details, testing arrangement, and instrumentation of beams

strain were of the A-9 type. Continuous recording of strains allowed measurements to be taken beyond the point of diagonal cracking and to ultimate load.

Near the end of the program, because of indications from the earlier tests, SR-4 gages were placed on the top surface of six beams to study the possible development of tension in the "compression" zone of the concrete. These gages were located 9½ in. outside the load points.

Testing procedure

The beams were tested in a 1,000,000-lb hydraulic compression machine. Fig. 1 shows the details of the testing arrangement. The reaction bearing plates were seated in high strength calcined gypsum mortar to assure full bearing, but each load-bearing plate bore on two ¼ in. thick leather pads which were placed astride the SR-4 gages located at the third points (see cross-sectional view in Fig. 1). Dial gages at the center and third points indicated beam deflections to the nearest 0.001 in. In the 3000 and 4500-psi beam series the loads were applied in increments of 2000 lb. The higher strength beams were loaded in increments of 5000 lb. At each increment, the load was maintained for a period sufficient to record deflections and to mark the progression of cracks. About 30 min were required for each beam test.

TEST RESULTS AND DISCUSSION

The compressive strength and modulus of elasticity of the shear beam concretes were determined from 6 x 12-in. cylinders cured similarly to the beams; one such cylinder was taken from each of the three batches that constituted a beam. Table 3 presents these properties along with a tabulation of steel percentage, steel modulus, modular ratio, and the standard k-value for 21 beams tested in this program. E_c is the secant modulus of elasticity of the concrete computed at a strain of 0.001 in. per in. This strain value is the approximate compressive strain in the outer fiber of the concrete at initial diagonal tension cracking, as indicated by the SR-4 gage measurements. The elastic moduli of Table 2 differ slightly from those of Table 1 which are secant moduli at $0.3\ f_c'$. The last two columns of the table give values of k, computed and measured. The computed values were determined from the standard formula:

$$k = [2np + (np)^2]^{1/2} - np$$

Measured values of k were obtained from strain gage measurements made at the center section of the beams and the relationship for linear distribution of strain:

$$k = \frac{e_c}{e_s + e_c}$$

The computed values of k were used to determine jd, required to establish nominal shear strengths. The excellent agreement between values of k computed by the two equations indicates that jd was determined reliably up to the initial diagonal cracking load of these beams.

Ultimate strength of test beams

At each increment of load, the development of cracking was carefully marked. Under the lighter loads the usual flexural cracking developed within the middle third of the beam span in the pure moment section. At higher

loads, diagonal tension cracks extended from under or just outside one of the load points to the vicinity of the reinforcing steel at a point approximately 1 ft from the reaction. These diagonal cracks formed suddenly in the lightweight beams and were accompanied by sharp reduction of testing machine load and large increase in deflection. The sand-and-gravel concrete beams, particularly those of lower concrete strength, developed diagonal cracking more gradually but the behavior pattern of load reduction and increased deflection was still evident. The load at first diagonal cracking was also the ultimate load for five of the beams tested. The other beams recovered the load that was initially lost and were able to absorb additional load to cause diagonal cracking at the opposite end. The same pattern of load reduction and increased deflection was repeated. All beams that sustained diagonal tension cracks at both ends failed at an ultimate load which was above that at first diagonal cracking. This amounted to less than 10 percent increase for two of the beams while the 14 remaining beams exhibited 50 to 100 percent increase.

TABLE 2—CONCRETE PROPERTIES USED IN CHECKING BEAM CROSS SECTION

Aggregate	3000-psi concrete		4500-psi concrete		7000-psi concrete		9000-psi concrete	
	E_c, 10^6 psi	n*	E_c, 10^6 psi	n	E_c, 10^6 psi	n	E_c, 10^6 psi	n
Sand and gravel	3.4	8.5	4.0	7.25	5.0	5.8	5.4	5.4
Lightweight	2.0	14.5	2.2	13.25	2.3	12.6	2.4	12.1

*n is the ratio of steel elastic modulus to that of concrete.

TABLE 3—PROPERTIES OF SHEAR BEAMS

Aggregate No.	Beam No.	28-day strength f_c', psi	Reinforcement ratio p	E_s, 10^6 psi	E_c,* 10^6 psi	n	Computed k	Measured k
9	9C	6910	0.05	27.5	2.89	9.5	0.61	0.61
2	2A	3680	0.025	27.5	1.57	17.5	0.60	0.60
2	2B	5350	0.025	27.5	1.81	15.2	0.57	0.60
3	3A	3310	0.025	27.5	1.87	14.7	0.56	0.56
3	3B	4090	0.025	27.5	1.98	13.9	0.56	0.59
4	4A	2980	0.025	27.5	1.53	18.0	0.60	0.56
4	4B	4890	0.025	27.5	1.83	15.0	0.57	0.56
4	4C	7000	0.05	27.5	2.09	13.2	0.66	0.68
4	4D	8160	0.05	27.5	2.26	12.2	0.65	0.65
5	5A	3490	0.025	27.5	1.90	14.5	0.56	0.53
5	5B	4790	0.025	27.5	2.20	12.5	0.54	0.53
6	6A	3670	0.025	27.5	2.25	12.2	0.53	0.52
6	6B	4870	0.025	27.5	2.61	10.5	0.51	0.49
7	7A-X	3210	0.025	27.5	2.02	13.6	0.55	0.52
7	7A	4240	0.025	27.5	2.23	12.3	0.53	0.53
7	7B	5200	0.025	27.5	2.54	10.8	0.51	0.49
8	8A-X	3700	0.025	27.5	2.97	9.3	0.49	0.49
8	8A	4020	0.025	27.5	3.13	8.8	0.48	—
8	8B	5380	0.025	27.5	3.59	7.7	0.46	0.49
8	8C	8410	0.05	27.5	4.30	6.4	0.54	0.54
8	8D	10,680	0.05	27.5	4.76	5.8	0.53	0.53

*E_c is secant modulus of elasticity of concrete computed at a strain of 0.001 in. per in. corresponding to beam maximum fiber strain at diagonal crack formation.

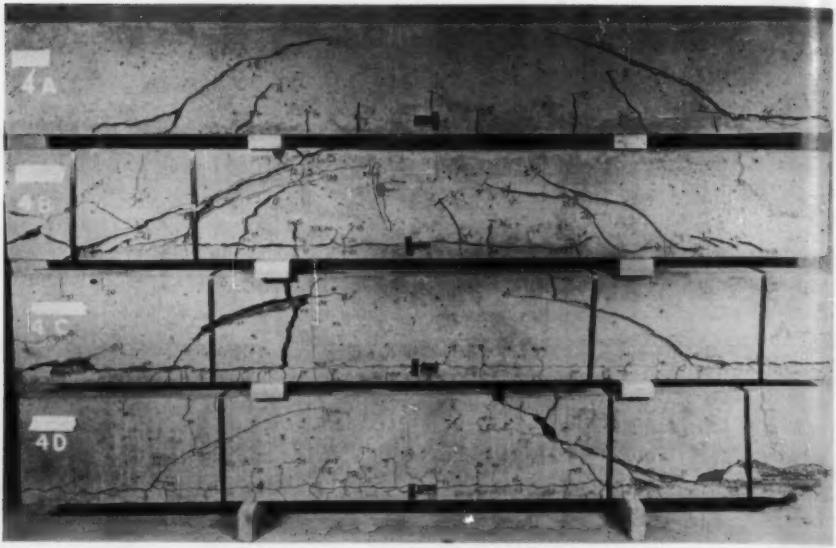

Fig. 2—Cracking of beams with Aggregate 4

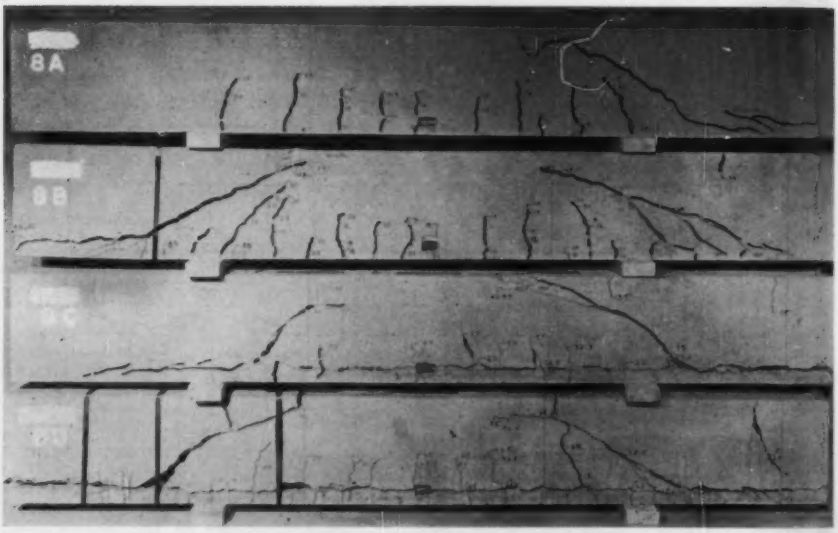

Fig. 3—Cracking of sand-and-gravel concrete beams

Fig. 2-5 show typical beams after completion of testing. As loading progressed above initial cracking load, the diagonal cracks in most cases continued to extend horizontally into the region of pure flexure. Other cracks formed near the ends, vertically if from the top of the beam and horizontally

if from the end. At the ultimate load, rupture occurred near a load point, with secondary splitting along the reinforcing steel in most cases.

Thus, at least two distinct types of failure occurred in this test series. One type was characterized by complete failure at the diagonal tension cracking load. The failures shown in Fig. 4 are typical. In the other type ultimate load was reached at levels considerably above diagonal cracking load, typified by the beams shown in Fig. 5. Crushing at a load point was generally noted in the latter type. In the former type of failure, vertical cracks approximately 9½ in. outside the load points appeared along with the diagonal crack but with no evidence of crushing. Strain gages at the load points indicated only 800 to 1600 millionths of strain, an amount far less than would normally be required for compression failure.

Moody, Viest, Elstner, and Hognestad[4] have pointed out that a redistribution of stresses takes place with the occurrence of diagonal tension cracking. This redistribution requires that the steel stress can no longer be distributed in accordance with the external moment, but must be such that the shear block outside of the diagonal crack is maintained in equilibrium. Fig. 6 shows forces on the shear block considered as a free body. Since no web steel exists, these forces are load, load reaction, steel tension, and the total force of concrete compression due to flexural action. In order that equilibrium may exist, the resultant of these forces must pass through the intersection of

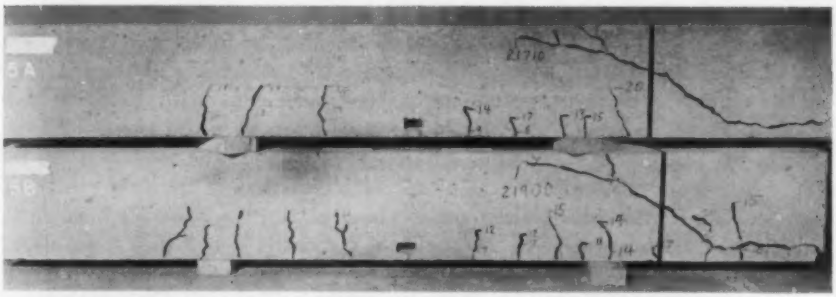

Fig. 4—Typical failures at diagonal cracking load

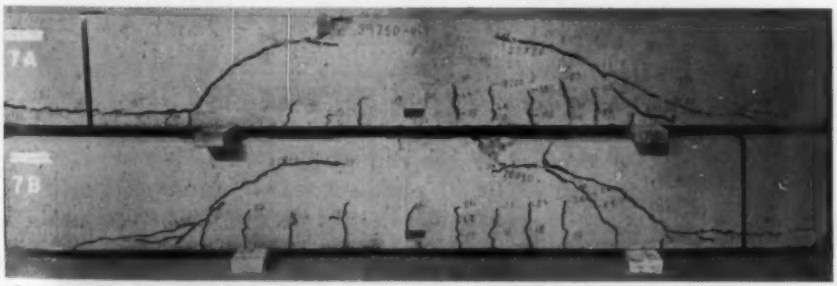

Fig. 5—Typical failures with ultimate above diagonal cracking load

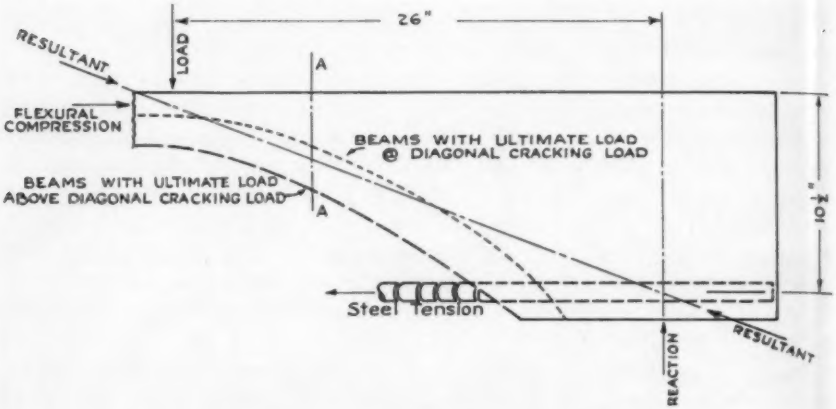

Fig. 6—Location of resultant of forces on shear block

the line of the reaction force with the line of the steel tension force and through the intersection of the lines of force due to load and to concrete compression. This latter point is somewhat indefinite due to unknown distribution of the compression stresses, but since diagonal cracking generally extends nearly to the top fiber of the beam, the location of the intersection shown in Fig. 6 is sufficiently well defined for present purposes. The two dashed lines indicate the average locations of diagonal tension cracking in these test beams, the line of short dashes for beams with ultimate load occurring at the diagonal cracking load and the line of longer dashes for beams with ultimate load above diagonal cracking load. Each line is an average of the measured locations of five cracks. It will be immediately evident from the relative positions of the diagonal cracks and the resultant of forces that in the first case very high tension would exist at the outer fibers of some section such as *A-A*, while in the second case, with the resultant force line well within the section, only relatively low tensile or even compressive stresses would occur.

Such a diagram clearly demonstrates that reinforced beams similar to those of this investigation can fail in direct tension or by compression at higher loads, depending on where the diagonal crack appears with respect to the resultant forces. It will be noted from Fig. 6 that the lines representing the two average locations for diagonal cracking are only a few inches apart. Thus, to predict accurately the location and accompanying type of failure would be impracticable. Indeed, a demonstration of this uncertainty of the mode of failure was provided by the performance of two of the sand-and-gravel concrete shear beams, No. 8A and 8A-X. The concretes were of equal compressive strength yet one beam failed at the load required to produce the first diagonal crack while the companion beam failed at a load appreciably above that which produced diagonal tension cracking as shown in Table 3. Apparently for these beams the critical location of diagonal cracking relative

to the resultant of applied forces on the shear block was a matter of pure chance, and as a consequence only the diagonal cracking load is of practical use as a measure of nominal shear strength.

As a matter of interest, it might be pointed out that several beams which sustained ultimate loads above the diagonal cracking load gave indications of final failure by direct tension. For example, Beams 4C and 4D (Fig. 2) and Beam 8D (Fig. 3) showed only vertical cracks near the load points with no evidence of crushing. This type of failure, of course, is possible only as long as the resultant of the forces falls outside the middle third of any section of the shear block.

A consistent excess of load capacity above that required to cause diagonal cracking has been observed primarily in beams having span-depth ratios less than that reported here. Cracking of such members may, in effect, convert the beam into a tied arch, with the resultant of the compressive forces falling well within the intact concrete section. Test beams of greater span-depth ratio have, in general, shown no excess load capacity after diagonal cracking.

Nominal shear strength

On the basis of the above conclusion that the ultimate load capacity of these shear beams cannot give dependable values of nominal unit shear, only the diagonal cracking loads were used in the computation of this factor. Table 4 presents the cylinder strength, diagonal cracking load as indicated by

TABLE 4—RESULTS OF SHEAR BEAM TEST

Beam No.	f_c', psi	P_{s1}, kips	P_{s2}, kips	P_U, kips	v_{s1}, psi	v_{s2}, psi	$\frac{v_s \text{ (average)}}{f_c'}$
9C	6910	26.00	28.40	50.80	259	283	0.039
2A	3680	19.20	19.20	37.00	190	190	0.052
2B	5350	17.97	21.18	43.22	176	208	0.036
3A	3310	19.80	—	19.80	193	—	0.058
3B	4090	19.82	20.25	22.00	193	197	0.048
4A	2980	18.45	20.81	24.71	183	206	0.065
4B	4890	21.56	22.00	37.69	211	216	0.044
4C	7000	24.40	28.00	49.10	249	285	0.038
4D	8160	28.25	28.25	53.50	287	287	0.035
5A	3490	21.71	—	21.71	212	—	0.061
5B	4790	21.90	—	21.90	212	—	0.044
6A	3670	19.75	20.00	20.05	191	193	0.052
6B	4870	19.47	21.56	31.55	186	206	0.040
7A-X	3210	25.46	—	25.46	248	—	0.077
7A	4240	27.72	27.90	39.75	268	269	0.063
7B	5200	28.04	29.80	39.40	269	285	0.053
8A-X	3700	24.06	28.05	36.16	228	266	0.067
8A	4020	25.95	—	25.95	245	—	0.061
8B	5380	28.50	33.70	40.64	267	316	0.054
8C	8410	37.74	40.00	57.24	365	387	0.045
8D	10,680	39.30	40.00	74.40	378	385	0.036

f_c' = cylinder strength of concrete.
P_{s1} = load causing diagonal crack at one end of beam.
P_{s2} = load causing diagonal crack at other end of beam.
P_U = ultimate load capacity of beam.
v_{s1} = nominal unit shear based on P_{s1}.
v_{s2} = nominal unit shear based on P_{s2}.

testing machine, ultimate load capacity, and nominal shear for all of the beams tested. Nominal unit shear v_s was computed as the standard value of total shear divided by bjd. Total shear is equal to half the machine load at diagonal cracking. In those tests in which diagonal cracks formed at both ends of the beam, both values are reported, thus giving a better estimate of the range of shear strength for any particular lightweight aggregate. A comparison of ultimate loads with diagonal cracking loads shows that the spread of nominal unit shear would be nearly doubled if computed on the final load basis. The writer feels that such practice has contributed to the wide range of shear values reported in the literature for beams without web reinforcement.

Fig. 7 presents the nominal unit shear strength as a function of concrete strength. Results for concrete made with lightweight aggregates No. 2-6 and No. 9 are grouped closely at approximately 75 percent of the shear strength of the normal weight concrete made with Aggregate 8. Aggregate 7, the sintered shale, produced concrete with a shear strength equal to that of Aggregate 8. No explanation of this increased strength relative to other lightweight concretes can be offered. However, in a later series of tests for ultimate load design coefficients, Aggregate 7 again showed behavior similar to the Elgin sand-and-gravel concrete. A search of the literature for comparable values of shear strength for lightweight concrete beams yielded only the tests of Haydite concrete by Richart and Jensen.[2] Their values for shear strength at comparable cylinder strengths are approximately 65 percent greater than those reported here. However, their values were computed from the ultimate load and their beam specimens were 1.6 times as deep in proportion to span as the beams of the present study.

Moody, et al.,[4] and Morrow and Viest[6] present the only values of shear strength found that have been computed on the basis of diagonal cracking load. The Moody beams were of sand-and-gravel concrete, the aggregate being designated as Wabash River sand and gravel. The tests, conducted at the University of Illinois, included Series A beams, 7 x 12 in. in section and 6 ft 3 in. long, loaded at the center point, and Series B beams, 6 x 12 in. in section and 10 ft long, loaded at third points. Steel percentages varied from 0.8 to 2.37 and concrete strengths from 880 psi to 5970 psi. The unit shears at diagonal cracking have been superimposed as small dots on Fig. 7.

Morrow and Viest[6] also used Wabash River sand and gravel for test beams, 12 x 16-in. in cross section with span varying from 42 to 240 in. Beams were loaded through a stub reinforced column cast integrally at the center of the beam. As in the Moody tests, no web steel was used. Longitudinal reinforcement varied from 0.57 to 3.83 percent and concrete strengths varied from 2000 to 6000 psi. Nominal unit shear strengths from the Morrow-Viest beams with a 70-in. span are also shown in Fig. 7 as four-point stars. To prevent further confusion on the diagram, values from the longer span Morrow-Viest beams were not plotted, but these also exhibit unit shear strengths of the same approximate magnitude.

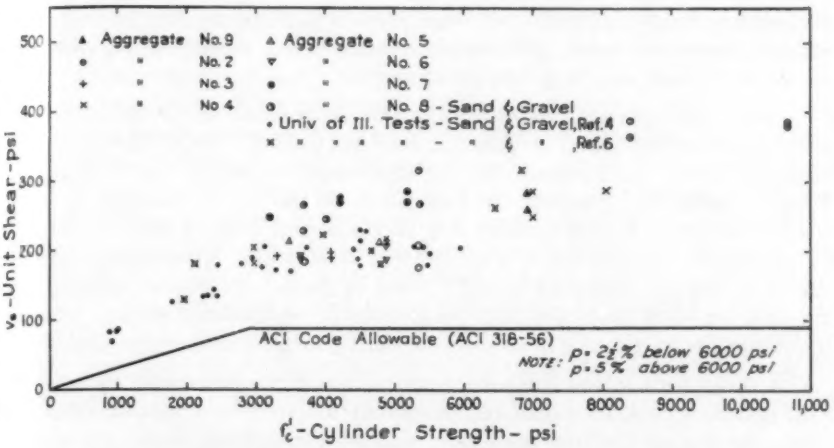

Fig. 7—Nominal shear strength as a function of cylinder strength

It is evident that at comparable strengths these sand-and-gravel concretes showed a nominal shear strength no greater than that of the lightweight concretes. For test conditions selected the shear values plotted in Fig. 7 have a minimum factor of safety of 2.0 above the working stresses given by "Building Code Requirements for Reinforced Concrete (ACI 318-56)," and in the higher strength ranges they exceed these working stresses by a factor varying from 2.9 to 4.0.

In using these data for the design of lightweight concrete structures, it should be remembered that the shearing resistance of beams is reduced by axial tension, and that with many lightweight aggregates the drying shrinkage is greater than with normal weight aggregates.[3] When such aggregates are used in members in which axial tension may develop as a result of restrained shrinkage, conservative values of allowable shear should be selected.

Measured strains

Voluminous strain data taken during loading of the shear beams proved of considerable value in establishing the type of failure and beam performance characteristics. The locations of gages are shown by Fig. 1. Gages 3 and 4, located at the center of the beam on the top fiber, and on the reinforcing steel, gave the "measured" location of the neutral axis in the pure moment zone. Gages 1 and 2, located on the steel 11 in. from the ends of the beams, showed the effects of stress redistribution due to the diagonal cracking and also indicated that bond failure did not contribute to either diagonal cracking or to ultimate failure. Since the bond stress at the end of the bar must be zero, the average bond stress in the length of 11 in. of steel reinforcing to the gage may be computed from the total force in the steel and the size of the bar. No bond stresses so computed exceeded the pull-out values reported by Shideler[3]

for bottom bars. In regard to bond stress computed by the conventional method, only one beam, 2B, exhibited bond stress exceeding that allowed by the ACI Code, and then only by 15 percent.

Gages 5 and 6 located under the third-point loads definitely indicated that those beams which failed at diagonal cracking load did not achieve sufficient strain to fail by compression. In the later stages of testing, Gages 5x and 6x were mounted 9½ in. outside the load points on the top of the beam to check the tensile strain shown to exist due to the eccentricity of the resultant of forces on the shear block. This location was chosen from observation of cracks in beams previously tested. None of these later beams failed at the diagonal cracking load, but large reductions in compressive strains, or even tensile strains were found in all cases as loads increased to the ultimate.

For those tests which included concrete strain measurements outside the load points, a calibrated load cell or electric dynamometer was mounted between the testing machine head and the load-distributing beam. Data from

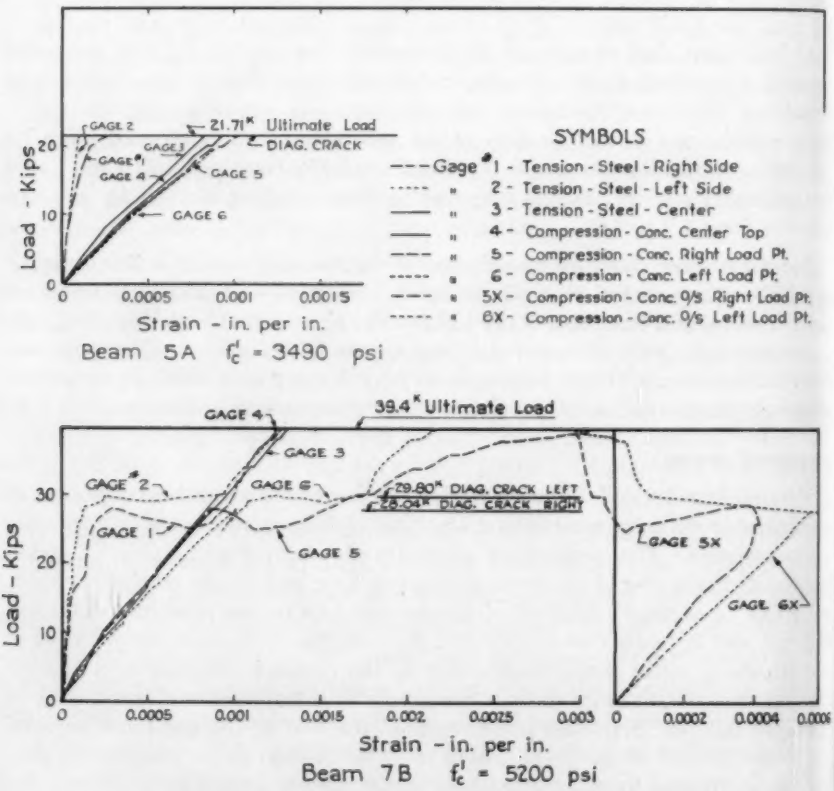

Fig. 8—Typical measured strains of lower strength beams

this cell were continuously recorded, and thus the load-strain relationship could be plotted during sharp load reductions at diagonal cracking. This strain history for Beams 5A and 7B appears in Fig. 8. From zero load to the load at diagonal cracking, symmetry of corresponding strains shows that the third-point loads were nearly equal. After diagonal cracking this similarity of strain distribution did not continue since the strains on the shear block were distributed in accordance with the position of the diagonal crack and with its proximity to the top surface of the beam. Beam 5A failed completely at the cracking load, but the upper diagram of the figure shows maximum compressive strain far less than that which would indicate a compression failure. Examination of Fig. 4 shows vertical cracking outside of the load point, caused by tension in the shear block referred to above. The right side of the lower diagram of Fig. 8 shows development of such tension in Beam 7B.

Deflections

Deflection of lightweight concrete beams is an important design consideration because of the relatively low modulus of elasticity of this material. Fig. 9 is a typical load-deflection diagram for these tests. Dial gages located at the third points and at the center again indicated that the equipment gave symmetrical loading even after the diagonal cracks. A rather large increase in deflection may be noted at each of the diagonal cracking loads. At lesser loads the curves are fairly linear and should be amenable to prediction.

Table 5 shows a comparison of computed and measured deflections for all the beams at loads less than those which caused diagonal cracking. The deflection of the "cracked" flexural section was computed by the "transformed

TABLE 5—COMPUTED AND MEASURED DEFLECTION OF SHEAR BEAMS

Beam No.	f_c', psi	Load, kips	Computed center deflection, in.	Measured deflection, in.		
				Left third point	Center	Right third point
9C	6910	20	0.057	0.058	0.064	0.057
2A	3680	16	0.087	0.072	0.093	0.081
2B	5350	16	0.081	0.076	0.085	0.070
3A	3310	16	0.080	0.080	0.091	0.082
3B	4090	16	0.078	0.074	0.088	0.078
4A	2980	16	0.088	0.086	0.099	0.089
4B	4890	16	0.081	0.077	0.088	0.077
4C	7000	20	0.067	0.061	0.070	0.064
4D	8160	20	0.066	0.059	0.061	0.058
5A	3490	16	0.079	0.071	0.081	0.072
5B	4790	16	0.074	0.069	0.078	0.068
6A	3670	16	0.077	0.068	0.081	0.072
6B	4870	16	0.069	0.066	0.076	0.068
7A-X	3210	16	0.077	0.070	0.079	0.069
7A	4240	16	0.074	0.061	0.074	0.067
7B	5200	16	0.070	0.063	0.076	0.071
8A-X	3700	16	0.065	0.069	0.081	0.078
8A	4020	16	0.064	0.057	0.067	0.058
8B	5380	16	0.061	0.048	0.055	0.050
8C	8410	20	0.047	0.042	0.048	0.043
8D	10,680	20	0.045	0.044	0.051	0.046

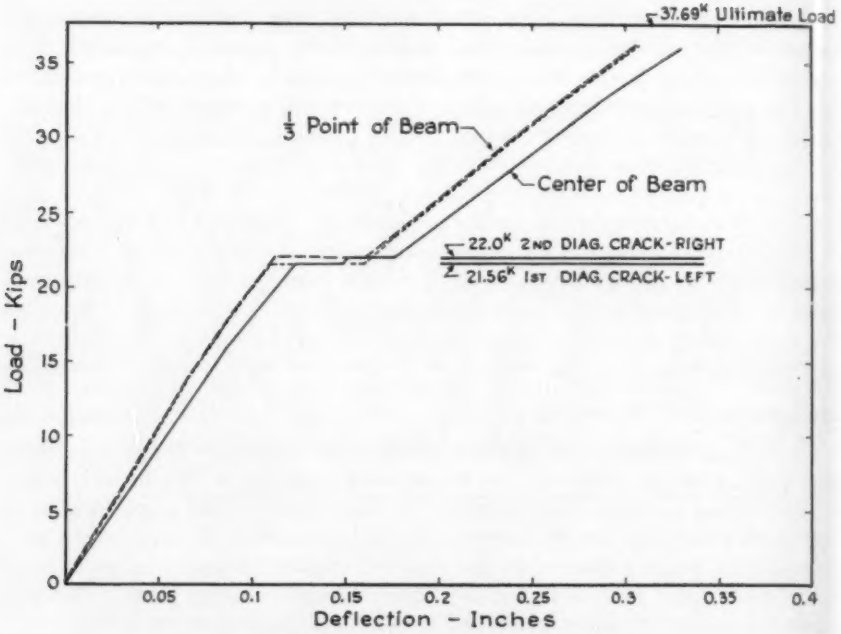

Fig. 9—Measured deflections, Beam 4B

cross section" method at the center of the beam using the actual characteristics found in Table 3. The computed deflection is in all but two cases less than the measured, and the deficiency averages 6 percent. The above method predicts only the deflection due to flexural action, in accordance with general practice.

Table 5 indicates an important consideration in the use of lightweight concrete for structural purposes. If the deflections of lightweight and normal weight beams of the same compressive strengths are compared, the deflections of the lightweight beams are from 15 to 35 percent greater than those of the sand-and-gravel beams. In an accompanying comparison, the modulus of elasticity (Table 3) of the lightweight concrete varies from two-thirds to half of that of normal weight concrete.

SUMMARY AND CONCLUSIONS

1. This investigation was limited to beams of one size and type of loading, with only one test of each of the eight aggregates at each compressive strength. Span was 6 ft 6 in., depth to steel was 10½ in., and loading was at third points.

2. The ultimate load of these beams without shear reinforcement, and failing in diagonal tension, was materially affected by the location of the diagonal crack. Therefore, only loads at diagonal cracking were used to com-

pare the nominal shear strengths. Ultimate loads above this cracking load were obtained only by chance and thus had no practical significance.

3. While the nominal unit shear strength of the lightweight concrete was lower than that of companion normal weight concrete beams, a comparison with normal weight concrete beams tested by other investigators shows comparable shear strength for concretes of the same compressive strength when used in unrestrained beams. In restrained beams, some lightweight aggregates may develop higher shrinkage stresses than normal weight aggregates.

4. All values of unit shear strength reported have safety factors of 2.0 to 4.0 above the ACI Building Code working stresses, the lower value being applicable to lower strength concretes.

5. The nominal unit shear strengths were similar for all types of lightweight aggregate with the exception of the higher values for Aggregate 7.

6. The deflections of both the normal weight and the lightweight beams after flexural cracking under working loads were computed by the "transformed cross section" method with acceptable accuracy. However, such computations should use realistic values of modulus of elasticity for the concrete.

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6. Morrow, JoDean, and Viest, I. M., "Shear Strength of Reinforced Concrete Frame Members Without Web Reinforcement," ACI JOURNAL, Mar. 1957, *Proc.* V. 53, pp. 833-870.

Discussion of this paper should reach ACI headquarters in triplicate by Dec. 1, 1958, for publication in the March 1959 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Development of the new AASHO specifications for composite steel and concrete bridges

I. M. Viest, R. S. Fountain, and C. P. Siess, *Bulletin* No. 174, Highway Research Board, 1958, pp. 1-17

The AASHO Committee on Bridges and Structures in 1955 prepared a new specification for composite bridges. The scope of the specification was restricted to structures composed of steel beams with concrete deck slabs connected by shear connectors.

The major new provisions are concerned with the design of shear connectors, the design of negative moment sections, and the effects of creep of concrete on the stress conditions in a composite beam.

Construction

Below grade construction

Building Construction Illustrated, V. 25, No. 1, July 1958, pp. 32-46

Discusses and pictures practice in the construction of foundations of various types, slabs on grade, basements, crawl spaces, retaining walls, etc. Unusually well illustrated, the article should interest estimators as well as designers and builders.

Winter construction

C. R. Crocker, *The Engineering Journal* (Montreal), V. 41, No. 2, Feb. 1958, pp. 43-49 (discussion, p. 77)
Reviewed by Aron L. Mirsky

A comparison of Swedish, Russian, and Canadian methods of winter concreting, and an outline of what could be done, particularly by architects and engineers, to reduce costs on this type of construction. Primary goal is more efficient utilization of labor without summer employment and winter unemployment peaks.

Design for a warped deck garage

E. M. Khoury, *Consulting Engineer*, V. 10, No. 1, Jan. 1958, pp. 109-111
Reviewed by Aron L. Mirsky

Interesting approach to the problem of large multideck parking garages: all decks are level except at center of structure, where they are warped in the form of a hyperbolic paraboloid which serves as the travel lanes between floors. Level exterior portions could even be used for offices, with parking space in core.

The import of concrete for modern construction (Die Bedeutung des Betons für das neuzeitliche Bauwesen)

Peter Misch, *Die Bauzeitung* (Stuttgart), V. 62, No. 9, Sept. 1957, pp. 426-428
Reviewed by Aron L. Mirsky

Brief but trenchant presentation of the uses to which concrete has recently been put (*e.g.*, tall TV tower, Stuttgart; Benjamin Franklin Hall, Berlin; and others), and the role of cooperation among architect, engineer, contractor, material man, and others concerned in attaining this stature and maintaining it in the future.

Dams

Experience on arch dam construction in Norway (in Norwegian)

Chr. F. Groner, *Nordisk Betong* (Stockholm), V. 2, No. 1, 1958, pp. 1-12
Reviewed by Margaret Corbin

Since 1930 arch dams in Norway have been designed as slender reinforced concrete dams with a cement content of 350-400 kg per cu m.

A part of copyrighted Journal of the American Concrete Institute, V. 30, No. 3, Sept. 1958, *Proceedings* V. 55. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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They are built with as few construction joints as possible; all the joints are dentated (serrated) to prevent seepage. The downstream side is protected with an insulating wall to avoid frost damage. Excellent experience with concrete dams has been recorded so far and they are considered superior to all other types of dams both with respect to safety factor and to statically correct force distribution.

Design

Steel stresses in reinforced concrete bridges

G. B. Colebridge, *Surveyor and Municipal and County Engineer* (London), V. 117, No. 3443, Apr. 19, 1958, p. 350

Highway Research Abstracts June 1958

As far as allowable working stresses in reinforcement are concerned, British practice has tended in the past to be more conservative than in most continental countries. Continental engineers have revised their ideas on the stresses to be used in bridge design and have worked to much higher steel stresses in many important bridges built since the war.

Table shows the maximum permitted stresses in high tensile steel in bridge structures in various European countries.

Design of folded plate roofs

Howard Simpson, *Proceedings*, ASCE, V. 84, ST 1, Jan. 1958, pp. 1508-1–1508-21

Author's Summary

The procedure for the analysis of a single-span folded plate structure is reviewed and discussed with the aid of an illustrative example. The effects of relative displacements of the longitudinal edges are considered.

South American meetings on structural engineering—official record of the fifth meeting (Jornadas Sudamericanas de ingenieria estructural—memorias de las quintas jornadas)

Faculty of Engineering of the University of Uruguay, Ministery of Public Works and Ministery of Education of Uruguay, Montevideo, Oct. 1954, 337 pp.

Reviewed by Celso A. Carbonell

The book describes 21 works presented during the October, 1954, meeting all of them dealing with scientific structural studies. Argentina presented one paper; Brazil seven; Chile five; and Uruguay eight. Each one of the works is an exhaustive study of a particular structural problem with a good number of illustrations and examples. A good addition to the library of a structural engineer.

Failure of reinforced concrete beams subject to bending and shearing stresses (La rupture par flexion et par effort tranchant dans les poutres en béton arme)

R. Chambaud, *Annales de l'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 110, Feb. 1957, pp. 169-205

Reviewed by Henri Perrin

In the first part the author summarizes his theory on failure by bending which he had already presented in previous papers. The second part deals with a new approach to explain failure by shear. Only simple beams of I-section and T-section are considered. It is suggested that the concrete struts under compression are more inclined on the horizontal than generally assumed. Various aspects of the problem are covered by new formulas, as for instance a lower limit for shear reinforcement. Comparisons between computed values and test results confirm the proposed theory. A few photographs of tested beams are also commented on.

Types and forms of shell structures

Milo S. Ketchum, Ketchum and Konkel, Denver, 1958, 122 pp.

Provides sketches and descriptions of the many variations possible with concrete shell structures. Purpose of the booklet is to aid the designer in selection of a structure for a particular project. Particularly appropriate for architects or structural engineers working for architects.

Analysis and tests of translational shells of composite concrete and masonry (Berechnung von doppelt gekrümmten Schalen und Versuche an solchen aus Tonhohlsteinen)

Oskar Gmuer, *Schweizerische Bauzeitung* (Zurich), V. 75, No. 22, May 31, 1958, pp. 329-335

Develops theory for analysis of the membrane stresses in translational shells followed by a method of calculating edge stresses. Compares theories illustrated by numerical example and then compares to test results on

composite shells of reinforced concrete and masonry. Shells are composed of light hollow tile laid up on templates and light shores and then concreted. The test load consisted of sand bags suspended beneath the shell with loading controlled by pneumatic inflated cushions.

Design of tilt up buildings

F. Thomas Collins, Know How Publications, San Gabriel, Calif., 1st Edition, 1958, 162 pp., $12.50

Since tilt up is basically job site prefabrication, maximum economy requires design and details suitable for the construction method and familiar to the construction trades. The book presents designs and details to utilize proven field procedures, as well as many proposed improvements including the use of prestressing. Most detail and design emphasizes resistance to wind and earthquake.

Course on experimental analysis of structures (Curso de analisis experimental de estructuras)

Julio Ricaldoni, Faculty of Engineering, University of Uruguay, Montevideo, Sept. 1957, 170 pp.
Reviewed by Celso A. Carbonell

Explains the different techniques in use for the experimental analysis of structures, not only in its theoretical aspect but also with comments about the scope and possibilities of each case and its practical applications. It is actually a textbook to be used by the civil and industrial engineering students in the University of Montevideo.

The author specifically avoids the details of construction and manipulation of equipment leaving this matter to the specialists, but he cites some references specially selected for this purpose.

Materials

Alkali reactions in concrete—General (Alment om alkali reaktioner i beton)

P. Nerenst, *Progress Report* A1, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1957, 44 pp., 12 D.kr. (25 percent discount may be obtained by ordering full series)

An introduction to the work of the Committee on Alkali Reactions in Concrete of the Danish National Institute of Building Research and the Academy of Technical Sciences. This report is meant to give Scandanavian readers an over-all impression of the nature of alkali reactions in concrete based mainly on literature in the English language.

The first sections deal with the economical and technological aspects of the durability of concrete as a whole. A brief historical review of the research carried out in the United States on alkali reactions is presented, as well as a short summary of test methods. The paper contains several references to other publications of the committee and a comprehensive bibliography.

Two previous *Progress Reports* were reviewed in the ACI Journal in May 1957, *Proc.* V. 53, p. 1099, and in May 1958, *Proc.* V. 54, p. 1015.

Classification of Danish flints etc. based on x-ray diffractometry

A. Tovborg Jensen, C. J. Wohlk, K. Drenck, and E. Krogh Andersen, *Progress Report* D1, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1957, 37 pp., 12 D.kr. (25 percent discount may be obtained by ordering the whole series)

Forty-two Danish flints and other siliceous rocks have been studied by quantitative X-ray diffractometric methods. Most of the specimens were types used as aggregate in field concrete. The results have been tabulated in order of decreasing percentage of α-quartz. The order adopted is presumably also one of increasing chemical reactivity.

Opal in the petrographic sense is shown to be not an "end member" in the silica-hydrous silica system, but to comprise a range of substances extending from (1) the completely glassy through (2) the partly platycryptocrystalline with cristobalitic atomic arrangement of increasing order to (3) a state where a

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considerable part of the silica is present as α-quartz.

It remains to be seen if the classification of flints and other siliceous rocks based on petrographic examination or on x-ray diffractometry is the more useful one for the understanding of alkali reactivity.

Occurring of flint and chert types in Denmark (Flintforekomster i Danmark)

HELGE GRY and BENT SONDERGAARD, *Progress Report* D 2, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and the Academy of Technical Sciences, Copenhagen, 1958, 63 pp.; English summary, pp. 50-63; 12 D.kr. (25 percent discount may be obtained by ordering full series)

Presents a petrographic description of the Danish Cretaceous cherts (flints) and other silicified rocks in Denmark.

Anhydrite cement (in Croatian-Serbian)

B. NESOVIC, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 1, Jan., 1958, pp. 34-39

Reviewed by J. J. POLIVKA

With reference to research of P. P. Budnikov (Russian Academy of Science), author thoroughly discusses properties and economy of anhydrite cements compared with typical lime and gypsum cements. Anhydrite, a mineral, differing chemically from gypsum in containing no water of crystallization, is found in great quantities in Bosnia and Herzegovina. Tests proved, e.g., that compressive strength of cement with 50 percent anhydrite, 3800 psi, increased to 5300 psi when 90 percent anhydrite was added. Manufacturing process of anhydrite cement is described.

Cement and concrete

ASTM *Special Technical Publication* No. 205, Jan. 1958, 152 pp., $4.00

A collection of papers presented during the Second Pacific Area National Meeting of the American Society for Testing Materials held in Los Angeles, Sept. 17, 1956.

Effect of storage on air-entraining cements

W. J. McCOY and S. B. HELMS, pp. 1-5 (discussion, p. 6)

AUTHORS' SUMMARY

Results of tests on samples of Types I, IA, IIA, and IIIA portland cement stored under various conditions for periods up to 5 years are reported. The effect of time and conditions of storage on air content, ignition loss, specific surface, and free lime results was determined at intervals during the 5-year period. These tests show that aging of cement does not cause any significant change in the above properties, or the setting time, or compressive strength if the cement is properly stored so as to prevent any reaction with moisture or carbon dioxide, as would be shown by no appreciable increase in the ignition loss.

Hydrophobic cement

U W. STOLL, pp. 7-15 (discussion p. 16)

AUTHOR'S SUMMARY

The resistance to atmospheric deterioration of fresh portland cement can be increased through the use of small amounts of interground oleic acid. Laboratory tests have demonstrated that the addition of from 0.1 to 0.4 percent oleic acid decreases the effort required during finish grinding of cement; that a more deterioration-resistant fresh cement will result; and that if the air-entraining tendencies of portland cement so treated are controlled, the resultant mortars and concretes are similar to those obtained using untreated cements. A suitable air-detraining agent is tri-n-butyl phosphate. This air-reducing agent may be either interground or added at time of mixing.

Carbonation of hydrated portland cement

GEORGE VERBECK, pp. 17-36

AUTHOR'S SUMMARY

Existing experimental and physicochemical evidence regarding carbonation of the constitutents of portland cement is reviewed and indicates that substantially all of the constituents of cement are subject to ultimate carbonation under ideal conditions.

Humidity during exposure to carbon dioxide appears to be a major factor influencing the shrinkage directly produced by carbonation. Concentration of carbon dioxide and size of specimen also have important effects, possibly produced by indirect changes in humidity within the specimem. Increased subsequent volume stability to moisture change was obtained by prior carbonation.

The influence of high pressure steam curing procedures on some physical properties of cement pastes containing siliceous additions is presented. The amount of water vapor absorbed at 36 percent relative humidity is a

measure of the amount and nature of the hydration product and is influenced by siliceous additions and decreased during curing at elevated temperatures. This fundamental characteristic of the hydration product is closely related to the elastic modulus, water-loss shrinkage, carbonation shrinkage, degree of carbonation, and water permeability of the paste.

The strict physicochemical significance of the phenolphthalein color test for estimating depth of penetration by carbon dioxide gas is questioned.

The partial replacement of portland cement in concrete

Bryant Mather, pp. 37-67 (discussion pp. 67-73)
Author's Summary

Sixteen replacement materials, including representatives of six classes—blast furnace slag, natural cement, fly ash, volcanic glass, calcined opaline shale, and uncalcined diatomite—were investigated together with five portland cements: two Type I cements, of low and high alkali content, a Type II, a Type III, and a Type IV. A total of 123 concrete mixtures containing crushed limestone aggregates up to ¾ in. in size, 6 ± ½ percent air, and having a slump of approximately 2½ in. were proportioned. About half the mixtures had a water-cement ratio of 0.5 by weight, to represent structural concrete, the remainder 0.8 to represent mass concrete. The 16 materials were used as partial replacements of portland cement in various percentages by solid volume from 8 to 70. The performance of the replacement materials in concrete was rated against selected values for certain properties regarded as critical for the two classes of concrete studied.

Alkali-aggregate phase of chemical reactivity in concrete. Part II—Deleterious reactions observed in field concrete structures

J. A. Hester and O. F. Smith, pp. 74-88 (discussion, pp. 89-90)
Authors' Summary

A report on an investigation of field structures showing effects of chemical reactivity between cement and aggregate. The history of each structure was examined to determine the source and type of cement and aggregate. Gels and affected aggregate particles from these structures were examined and analyzed. Concrete containing pozzolans was examined, including a chemical investigation of reactions of pozzolanic materials.

Effect of aggregate quality on resistance of concrete to abrasion

F. L. Smith, pp. 91-105 (discussion, p. 106)
Author's Summary

Sixty concrete mixes using different combinations of aggregate, combining good quality sand and gravels with poor quality sand and gravels, different sand contents and water-cement ratios, the latter ranging from 0.32 to 0.70, were used to determine their effect on the resistance of concrete to abrasion. Aggregates were selected which represented Los Angeles abrasion losses at 500 revolutions ranging between 12.6 and 39.1 percent. Measure of resistance to abrasion loss was determined by three types of abrasion machines: the Davis steel ball and the dressing wheel abrasion apparatus, which represent the types of wear caused by steel wheels under heavy load, and the Ruemelin shotblast apparatus, which simulates the erosion caused by solids in flowing water.

Abrasion resistance varies directly with both compressive strength and cement content and inversely with water-cement ratio, regardless of aggregate quality or combinations of aggregate used. No significant correlation was found to exist between the quality of the coarse aggregate as determined by either the sodium sulfate soundness or the Los Angeles abrasion test and the resistance to abrasion of concrete containing these aggregates.

Tests of prestressed expanded-shale concrete beams subjected to short-time and sustained loads

Raymond E. Davis, G. E. Troxell, D. O. McCall, and Ray A. McCann, pp. 107-130 (discussion pp. 130-134
Authors' Summary

This paper gives the results of tests on 19 flexural members of prestressed concrete, of lengths ranging from 24 to 65 ft and including beams of double-T section for floors and roofs, box beams for floor systems of bridges and buildings, and girders and purlins of I-section for roofs. Most of the members were composed of expanded shale lightweight concrete, and the steel was pretensioned. Most of the tests were made under loading which would produce failure in bending;

others in shear. Observations were made on all flexural members to determine deflections and cracking, and on selected beams to determine longitudinal strains in flexure or strength in shear. Auxiliary tests were made to determine compressive strength and modulus of elasticity of concrete, creep of concrete under sustained compressive stress, and loss of tension in steel reinforcement due to creep.

In general, the tests confirm the assumptions commonly made in design of prestressed concrete beams.

A physical method for determining the composition of hardened concrete

MILOS POLIVKA, J. W. KELLY, and CECIL H. BEST, pp. 135-152

AUTHORS' SUMMARY

This paper describes a "point-count" method of determining the cement content of hardened portland cement concrete by analysis of a cut section with a traveling microscope. The voids and aggregate content are estimated by direct observation; the cement content is then determined by difference, with correction for nonevaporable water in the cement paste. Sand-aggregate ratio is also computed.

Comparisons are made with results of chemical analysis according to ASTM C 85-54 and two other chemical methods developed during the investigation. The precision of each method is estimated, and the advantages and limitations are discussed.

Optimum amounts of chlorides added to concrete mixes placed in pavements at temperatures below freezing (in Russian)

V. I. OVCHAROV, *Beton i Zhelezobeton* (Moscow), No. 2, Feb. 1958, pp. 55-60

Reviewed by D. WATSTEIN

Recent studies of "cold" concreting showed that additions of calcium and sodium chlorides up to 16 percent by weight of mixing water have resulted in adequate strength development of concrete placed at from −5C to −10C, without preheating or any protection. The author stresses the need for the concrete to develop no less than 50 percent of its design strength within the period of "cold" cure at specified temperatures before the concrete is permitted to freeze at appreciably lower temperatures. Under this condition of initial "cold" cure, the compressive and transverse strength of the concrete showed a progressive increase up to an age of 360 days. The amounts of chlorides varied in the following amounts: 12 percent of NaCl, 8 percent NaCl + 4 percent $CaCl_2$, 12 percent NaCl + 2 percent $CaCl_2$, and 12 percent NaCl + 4 percent $CaCl_2$. In some cases the concretes containing $CaCl_2$ developed higher strengths at 360 days than the control specimens. The proportions of the concrete were 1:1.9:3.7 with $W/C = 0.45$. The specimens were 15 x 15 x 75-cm. prisms and the compressive strengths were determined on modified cubes.

Insoluble residue determination in portland and portland-slag cements

W. J. HALSTEAD and BERNARD CHAIKEN, *ASTM Bulletin*, No. 229, Apr. 1958, pp. 60-61

The ASTM method of analysis for insoluble residue in cement was found to give variable results depending upon the analyst's interpretation of the specified conditions. The authors suggest modifications and clarifications to increase the reproducibility of the test results. They compared the federal method with the ASTM procedure and obtained different results for certain interpretations of the latter procedure.

Characteristics of special aluminous cement (in Japanese)

RANZO NAGANO, *Semento Gijutsu Nenpo*, No. 3, 1949, pp. 52-58

CERAMIC ABSTRACTS
May 1958 (Y. Suzukawa)

To elucidate the characteristics of special aluminous cement prepared and patented by Hashimoto and Nagano, four compounds of the system CaO-Al_2O_3 were prepared from pure chemical reagents. The setting time and the strength developed by mortar and concrete were determined for all compounds except $3CaO \cdot Al_2O_3$.

Tensile tests of prestressing steel at elevated temperatures and evaluation of fire tests on prestressed and nonprestressed reinforced concrete members (in German)

DANNENBERG, DEUTSCHMANN, and MELCHIOR, *Proceedings*, Deutscher Ausschuss für Stahlbeton (Berlin), V. 122, 1957, pp. 9-35

Reviewed by PATRICK E. MURPHY

Tensile tests of prestressing wires (0.2-in. diameter) and bars (1-in. diameter) heated to

300 C, 500 C, and 600 C are reported. The results of these were combined with previous German fire tests on reinforced concrete members and some recent English and Dutch fire tests with prestressed concrete to form the basis for prediction of effect of fire damage to prestressed concrete members.

Pavements

Provisional roads made of prefabricated reinforced concrete slabs (in Polish)

S. Nowak, *Przeglad Budowlany* (Warsaw), No. 11, 1956, pp. 450-451

Polish Technical Abstracts No. 4 (28) 1957

The surface of provisional roads is laid with transportable reinforced concrete slabs of dimensions: 3 x 2.5 x 0.17 m, the weight of one slab amounting to 3020 kg. The slabs are laid in position by means of a truck crane. The surface of prefabricated slabs is smooth and they are much more durable than broken stone. A much higher degree of free passage for automotive traffic is ensured, irrespective of atmospheric conditions.

Prestressed Concrete

Advanced prestressed concrete design

Engineering Progress at the University of Florida, V. 12, No. 4, Apr. 1958; *Bulletin* 96, Florida Engineering and Industrial Experiment Station, 140 pp. $3

A compilation of papers presented at the Second National Prestressed Concrete Short Course, Jan. 27-31, 1958. The book contains 12 papers which include: an introduction to methods of prestressing and elastic design; five papers on materials uesd in prestressed concrete; and separate papers on composite beams, choice of sections and camber, bridge design, ultimate strength design, long span designs, unusual design examples, and continuous bridge beam design.

Approaches to street bridge over the River Glomma at Fredrikstad, Norway (in Norwegian)

A. G. Frandsen, E. Kalhauge, Chr. Ostenfeld and E. Stromme, *Nordisk Betong* (Stockholm), V. 2, No. 1, 1958, pp. 13-44

Reviewed by Margaret Corbin

Paper describes concrete approaches, 500 m long, to the main 196-m steel bridge span. The approaches consist of three continuous sections with spans of 23 m each. Precast prestressed concrete main girders are supported on cast-in-place columns with a mean height of 19 m (individual column height varies from 6 to 32 m). Each girder weighed 60 metric tons. After prestressing the beams were hoisted to the top of the columns, placed on flat jacks for vertical adjustment, and rendered continuous by local prestressing over the columns. A special steel bridge, used in erecting the main girders, was also used, in structural cooperation with the prestressed girders, to carry the formwork for the bridge slab, whereupon it was moved to the next section.

Transfer of prestress from pretensioned tendons to the concrete (in Russian)

E. G. Ratz, M. M. Kholmyansky, and V. M. Kolner, *Beton i Zhelezobeton* (Moscow), No. 1, Jan. 1958, pp. 4-13

Reviewed by D. Watstein

The authors present the results of an experimental and an analytical study of the problem of anchorage of pretensioned reinforcement. The test specimens were prisms ranging in cross section from 4 x 4 cm to 8 x 8 cm, and from 130 cm to 700 cm in length. The tendons consisted either of individual wires of 3, 4, and 5 mm diameter, or of two-wire (2.6 mm) and seven-wire (1.6 mm) strands. The individual wire tendons were deformed by means of indentations of variable depth and spacing. The strength of concrete at the time of transfer of prestress varied from 200 to 500 kg/per sq cm, and the level of prestress ranged from 0.04 to 0.60 of the cube strength of the concrete.

Extensive measurements of the distribution of slip near the ends of the tendons were made by means of optical comparators mounted at various sections of the concrete. The concrete was cored out at those sections to permit observation of the steel. Measurements made over long periods of time indicated that for wires having indentations, slip of the tendons ceases to increase after 30 to 40 days. From the observed values of slip, the required anchorage lengths were estimated for concretes of different strengths and for different types of reinforcement stressed from 2000 to 12,000 kg/per sq cm. A substantial reduction in the required anchorage length was noted as the concrete strength was increased. For tendons stressed to 10,000 kg/per sq cm, the required anchorage was reduced by a factor of from

2 to 5 as the concrete strength increased from 200 to 500/ kg per sq cm.

In the theoretical treatment of the problem, equations were developed for the distribution of slip within the anchorage length. Expressions are also given for the relationship between the stress in the reinforcement and the slip of the tendon with respect to concrete. The expression for the stress within the anchorage length shows that the distribution of stress changes with the level of stress and the diameter of the wire.

Water tower in the town of Orebro, Sweden (in Swedish)

ERLING REINIUS and KURT ERIKSSON, *Nordisk Betong* (Stockholm), V. 2, No. 1, 1958 pp. 45-72
Reviewed by MARGARET CORBIN

The water tower in Orebro, Sweden, has an unconventional shape, a volume of 9000 cu m and a total height of 58 m from ground level. Elevator shaft and stairway lead to a restaurant housed on top. The major part of the outer reservoir wall is a horizontally prestressed conical shell. The outer face of the wall is polygonal with 32 faces, every other one projecting 20 cm in order to provide anchorage for the prestressing cables. The lower part of the wall has a double curvature zone so as to furnish a smooth transition from the conical shell to a footring on top of the supporting cylindrical structure. Radii were chosen to produce moderate boundary moments and shearing forces, so that no prestressing was needed in the transition zone. Shell analysis is given. A model of the reservoir was made and tested for internal water pressure. Prestressing operations and erection of cast-on-the-ground tank by means of 32 hydraulic jacks are described.

Scientific principles of the application of natural stone in prestressed structures (in Polish)

J. PIETRZYKOWSKI, *Inzynieria i Budownictwo* (Warsaw), No. 12, 1956, pp. 457-466
POLISH TECHNICAL ABSTRACTS No. 3 (27) 1957

From the economic point of view (quarrying and hewing costs, transport, equipment, etc.), and assuming a definite working level of technology and standardization of production processes, stone may successfully compete with reinforced concrete in certain types of prestressed constructions. Detailed experiments were made with a view to solving the appropriate technical problems, and the results obtained were then compared with similar data obtained from abroad. Particular attention was paid to the problem of joints between stone elements carrying the prestressing forces as well as to that of permissible tolerances. It appears desirable that in such tests a safety coefficient for every individual case should be applied consisting of a number of coefficients characteristic of the stone properties as well as of individual working conditions of a given structure.

Experiments with prestressed concrete piles for deep water foundations in Lake Maracaibo (in Spanish)

GUILLERMO JOSE PADRON R., *Revista del Colegio de Ingenieros de Venezuela* (Caracas), No. 261, Dec. 1957, pp. 13-23
Reviewed by JOSEPH J. WADDELL

The author describes design and construction of prestressed concrete piles. Piles were pointed, hollow, and were cast in a horizontal position, using conventional reinforcing plus tensioned cables. The piles were tested as follows:

1. A 33-ft pile was tested in flexure to failure.

2. A 60-ft pile was driven into hard ground to test the strength of the head of the pile while driving.

3. A 145-ft pile was tested in flexure to determine the maximum moment under a limited load, then driven, in deep water, into hard ground, treating it in the same manner as piles driven at actual sites in deep water.

After the pile had been driven, its resistance to flexure and deflection was tested by applying a horizontal force at the butt of the pile until failure occurred.

Considerable space in this well illustrated article is given to discussions of design criteria and methods of construction.

Grout used for prestressed concrete (in German)

KURT WALZ, *Zement-Kalk-Gips* (Wiesbaden), V. 10, No. 2, 1957, pp. 53-56
CERAMIC ABSTRACTS June 1958 (Hartenheim)

The essential properties of grout are sufficient fluidity, low shrinkage, and no water separation. The water-cement ratio in the

preparation of grout is usually 0.36 to 0.44, but it should be as low as possible to give high strength and early resistance to frost. Agents added to increase fluidity should not contain $CaCl_2$. Testing methods are described.

Pressure grouting for prestressed concrete (Einpressmörtel für Spannbeton)

Reprinted from *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 12, Dec. 1957 and V. 53, No. 1, Jan. 1958, 19 pp.

A collection of five papers which present building code requirements for pressure grout, new developments in code requirements for pressure grouting, testing of materials and grout, shrinkage and volume change of grout, and new equipment for grouting.

Properties of Concrete

Frost resistance of mass concrete

J. Orth, *Schweizer Archiv für angewandte Wissenschaft und Technik* (Solothurn), V. 23, No. 6, 1957, pp. 170-88; and No. 7, 1957, pp. 215-31

Ceramic Abstracts
June 1958 (Hartenheim)

Amount of cement, cement-water ratio, granulometry, and aggregates are important in concrete subjected to extreme and rapid changes in temperature. Their influence on the physical behavior of concrete is shown.

Discussion on the ultrasonic testing of concrete

R. Jones, and others, *The Structural Engineer* (London), V. 35, No. 5, May 1957, pp. 190-206
Reviewed by C. P. Siess

Seminar relating to techniques and applications of pulse-velocity technique. Several contributions mentioned unreliability of velocity-strength correlation although velocity-modulus correlation was good. Also several references to need and difficulty of measuring length of path. In spite of these limitations numerous practical applications in laboratory, plant, and field were cited including: quality control of concrete in both precast and cast-in-place construction, investigations of structural adequacy, determination of setting time, location of cracks, and others.

Final note by C. F. Clarke describes use of gamma radiography using radiocobalt to locate reinforcement and to determine positions and sizes of areas of low density, and presumably, low strength concrete.

Effects of cement admixtures on electric corrosion of reinforced concrete (in Japanese)

Yasuo Kondo, Akihiko Takeda, and Setsuji Hideshima, *Journal*, Japan Society of Civil Engineers, V. 43, No. 2, Feb. 1958, pp. 1-8
Reviewed by Kiyoshi Okada

Describes experimental research on the effects of admixtures, $CaCl_2$ with and without some calcium lignosulfonates, on the electric corrosion of reinforcing steel embedded in 15-cm cube concrete specimen. Reinforcement is subjected, as anode or cathode, to direct current of 5, 10 and 20 volts, or to alternating current of 20 volts, each for 28 days, 3 hr each day. Concludes that $CaCl_2$ less than 1 percent of cement by weight gives no harm under direct current of less than 10 volts, and, when calcium lignosulfonates used together, the corrosion tendency is alleviated considerably even under direct current of 20 volts. Twenty volts alternating current causes no corrosion for all concrete containing the admixtures.

Creep in concretes

R. Dutron, *Bulletin* No. 34, International Association of Testing and Research Laboratories for Materials and Structures (RILEM), (Paris), 1957, pp. 11-33

A discussion of creep and shrinkage in concrete under various conditions of loading and curing for concretes of various compositions. Includes a bibliography of 20 recent references.

Shrinkage, creep, and compaction of high-strength concrete: Concrete of minimum shrinkage and creep (in German)

G. Balazs and J. Killian, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 17, No. 1-2, 1957, pp. 113-142

Hungarian Technical Abstracts
V. 10, No. 1, 1958

The evaluation of shrinkage, creep, and compaction tests on high-strength concrete has led to the conclusion that the age of the concrete at loading is the most important factor affecting the magnitude of shrinkage and creep. Under continuous load "compaction" occurs in concrete structures in compression. As a result of this compaction,

strength and modulus of elasticity increase. The earlier the concrete is loaded, the greater the compaction.

Radiation shielding concrete (in Danish)

ERIK TRUDSØ, *Ingeniøren*, B Edition (Copenhagen), V. 67, No. 7, Apr. 1, 1958, pp. 239-256
Reviewed by JESPER STRANDGAARD

The article is intended as a brief digest of representative literature about radiation shielding with concrete. Part 1 contains a summary of related physical definitions and terminology, and in Part 2 a synopsis is given of the technology of concrete shields, the choice of aggregate and additives, the proportioning of mix, structural shape selection, and the methods of placing the concrete. Twenty-one different concrete mixes with unit weights ranging from 2450 to 6560 kg per cum, and three methods of placing the concrete (conventional, prepacked, puddled) are tabulated together with resulting mechanical properties.

Structural Research

Transfer of high local pressure in reinforced concrete (in German)

WOLFGANG POHLE, *Proceedings*, Deutscher Ausschuss für Stahlbeton (Berlin), V. 122, 1957, pp. 37-52
Reviewed by PATRICK E. MURPHY

The transfer of high local compressive forces on reinforced concrete was studied in full scale tests of steel piles embedded in concrete caps. The light steel reinforcing hoops provided in the cap proved entirely adequate for resisting the bursting stresses.

Symposium on the observation of structures (in French and English)

Transactions of the RILEM Symposium in Lisbon, 1955, published by Laboratório Nacional de Engenharia Civil, Lisbon, 1957; V. I, 228 pp., and V. II, 1028 pp., $10.50

Volume I discusses the general problems, procedures, and equipment for observing behavior of structures in the field. Texts of 138 discussions of symposium papers are included, along with author index, and an index of structures cited in the various papers.

Volume II contains the 54 papers which deal with the following subjects: description and characteristics of instruments for measuring (a) displacements, joint openings, cracks and rotations, and (b) strains, stresses, forces, temperatures, and moisture; evaluation of the properties of the materials; and programs, descriptions, and interpretation of results of observations of dams, bridges, and other structures. New apparatus and observation methods as well as improvements of others are described. Important structures built during the last 10 years are studied from the standpoint of their behavior during and after construction.

Papers of particular interest in the field of concrete will be reviewed individually in later "Current Reviews" sections.

Model study of a dynamically laterally loaded pile

ROY D. GAUL, *Proceedings*, ASCE V. 84, SM 1, Feb. 1958, 33 pp.
HIGHWAY RESEARCH ABSTRACTS May 1958

A dimensionally scaled model of a vertical pile in soft soil has been dynamically tested. Results indicate that a low frequency oscillatory lateral load induces pile bending moments which closely correspond to moments caused by the same load applied statically. Analytical computation of pile moments based on the assumption of a soil modulus constant with depth appears to agree well with dynamic test results.

Composite beams with stud shear connectors

BRUNO THURLIMANN, *Bulletin* No. 174, Highway Research Board, 1958, pp. 18-38

Testing of a composite beam structure 32 ft long is described. The shear connectors on one of the beams were ¾-in. diameter studs with upset heads, welded to the top flange. On the other beam, ½-in. diameter studs with end hooks at right angle were used.

Both types of shear connectors behaved satisfactorily under all testing conditions. Tentative design recommendations concerning the use of stud shear connectors are made.

Concrete walls in compression under short-term axial and eccentric loads

A. E. SEDDON, *Publications*, International Association for Bridge and Structural Engineering, (Zurich), No. 16, 1956, pp. 457-468
AUTHOR'S SUMMARY

Tests were performed on full size walls and model walls and loads were applied over all or part of the wall length to include cases of distributed and concentrated loads. Infor-

mation is given on the effects on wall strength of varying the slenderness ratio and length of the wall, the strength of the concrete, the eccentricity and concentration of the applied load, and on the contribution of reinforcing steel to wall strength.

Determination of concrete stresses via strain measurements (Spannungsermittlung an Beton durch Dehnungsmessungen)

GUSTAV WEIL, *VDI Zeitschrift* (Düsseldorf), V. 99, No. 36, Dec. 21, 1957, pp. 1781-1786
Reviewed by ARON L. MIRSKY

Concise survey of two principal topics: effect of properties of the concrete (age, strength, elastic modulus, etc.), and types of apparatus (mechanical, electronic).

General

Proceedings, ASTM, V. 57

American Society for Testing Materials, Philadelphia, 1957, 1430 pp., $12.00

Sixty-six reports of technical committees together with appendices and 52 technical papers and discussions on a wide variety of subjects pertaining to research and testing materials.

All symposia and other special sessions published separately as *Special Technical Publications*, and all papers published in the *ASTM Bulletin* are listed.

An important adjunct is a subject and author index to all papers published in any form by the society in 1957. In addition to the reports and technical papers, the *Proceedings* contains much discussion not previously published.

Materials and methods of architectural construction

HARRY PARKER, CHARLES M. GAY, and JOHN W. MACGUIRE, John Wiley & Sons, Inc., New York, 3rd Edition, 1958, 724 pp., $12.00

A revision intended to bring the second edition (published in 1943) up-to-date. The book is eminently suitable as a text book for introductory undergraduate courses in architecture. The principal divisions are: materials of construction, engineering mechanics, and structural design and details applied in separate chapters for various materials of construction. Not suitable as a reference for practicing engineers or architects. Most of the new material added is superficial and obsolete older material is still included.

Proceedings of the symposium on safety of structures

Science Council of Japan, Architectural Institute of Japan, and Japan Society of Civil Engineers (joint publishers), Tokyo, Dec. 1956, 128 pp.

Presents 15 papers on various aspects of safety of structures. Papers include philosophical discussion of the concept of safety factors, test measurements of stresses in railway bridges, analyses of the effects of variations in the strength of both reinforcement and concrete on load carrying capacity of a reinforced concrete structure, variations in strength of steel, variations in quality of concrete as manufactured and as placed, and discussions of the Japanese code for earthquake safety. Individual papers of particular interest in the field of concrete will be reviewed later.

Applied mathematics for engineers and physicists

LOUIS A. PIPES, McGraw-Hill Book Co., New York, 2nd Edition, 1958, 725 pp., $8.75

Designed for the general advanced mathematics course offered to applied scientists, covers a wide range of topics in the advanced calculus fields. Gives the engineer and applied physicist the principal mathematical techniques they need to analyze the usual mathematical problems that arise in practice and to understand important current technical papers.

Scientific French

WILLIAM N. LOCKE, John Wiley & Sons, Inc., New York, 1957, 112 pp., $2.25

Scientific German

GEORGE E. CONDOYANNIS, John Wiley & Sons, Inc., New York, 1957, 165 pp., $2.50

Brief books for self instruction or classroom use to develop a reading knowledge of scientific and technical German and French. They are intended to be used in conjunction with a dictionary only. The books essentially consist of a bird's-eye view of the structure of the two languages with different peculiarities and patterns explained in detail as a reference grammar. Suitable for either beginners or those who need to refresh their knowledge of the languages.

Use of the Swiss hammer for estimating the compressive strength of hardened concrete

WILLIAM E. GRIER, *Public Roads*, V. 30, No. 2, June 1958, pp. 45-50

To determine the value of the Schmidt rebound hammer as a tool for estimating the strength of concrete used in highway construction, three series of laboratory tests were made as well as numerous associated studies. The hammer was also used to estimate the strength of several concrete structures in the field: beam sections cast for post-tensioning in a bridge; bridge piers about 2½ years old; a handrail on a bridge about 38 years old.

Results of the various tests show that factors such as surface smoothness, surface moisture condition, and type of coarse aggregate affect the strength values obtained by use of the device.

Reinforced concrete bin linings hardened by means of cast iron grit and shavings (in Polish)

W. GACA and S. BASTIAN, *Materialy Budowlane* (Warsaw), No. 11, 1956; pp. 359-362, No. 12, 1956, pp. 391-395

POLISH TECHNICAL ABSTRACTS No. 3 (27) 1957

Linings of rock material bins must be resistant to abrasion and also sufficiently strong to resist impact. Both these conditions, not always found together, are fulfilled by linings hardened by means of cast iron grit and shavings. Samples were subjected to tests—including hammer and milling tests—as regards resistance to abrasion, and strength On the basis of results obtained, the following conditions were determined: grain size, amount of grit to be added, the most suitable composition of vertical wall linings, and work method.

Reinforced concrete in architecture

ALY AHMED RAAFAT, Reinhold Publishing Corp., New York, 1958, 240 pp., $15

The author states that new developments in structural engineering for reinforced concrete are opening a gap between architecture and engineering in this field. The book is an attempt to bridge the gap in nontechnical language explaining the impact of new engineering principles upon architectural design. It begins with a concise historical survey of the early approaches to reinforced concrete including contributions by American engineers and then develops discussion on the architectural implications; in particular, of thin shell construction of all types. The discussion is made specific by numerous illustrations, drawings, and photographs of the principles involved and their applications in practice on outstanding structures throughout the world. Includes a 4-page bibliography. The book should be particularly useful and inspiring to creative architects and structural engineers dealing with such architects.

Manual of accident prevention in construction

Associated General Contractors of America, Inc., Washington, D. C., 5th Edition, 1958, 300 pp., $3.25

Consists of recommendations for accident prevention organization and individual chapters presenting recommended safe practices for various phases of construction work including concrete work. Primarily intended for use by contractors, the loose leaf manual will also be valuable as reference material to engineers preparing specifications or engaged in field inspection.

Sequential analysis applied to quality control of concrete

M. GREGORY, *Commonwealth Engineer* (Melbourne), V. 45, No. 9, Apr. 5, 1958, pp. 51-54

Presents a control chart on which the results of compression tests may be plotted during progress of a job. Average strength and control are clearly shown on the chart and trends are easily recognized leading to savings in cement for efficient control.

Observations on the Mexico City earthquake of July 28, 1957

J. J. GOULD, *Architect and Engineer*, Feb. 1958

AUTHOR'S SUMMARY

Discusses earthquake damage observed in Mexico City for each of the usual building types. Concludes that overturning is not a major problem because earthquake movements are too rapid. There is merit in the use of relatively flexible moment-resisting frames and stiff walls, and the lateral force requirements in the San Francisco code are adequate although the new code for Mexico City sets up lateral force requirements in some cases two to three times as severe.

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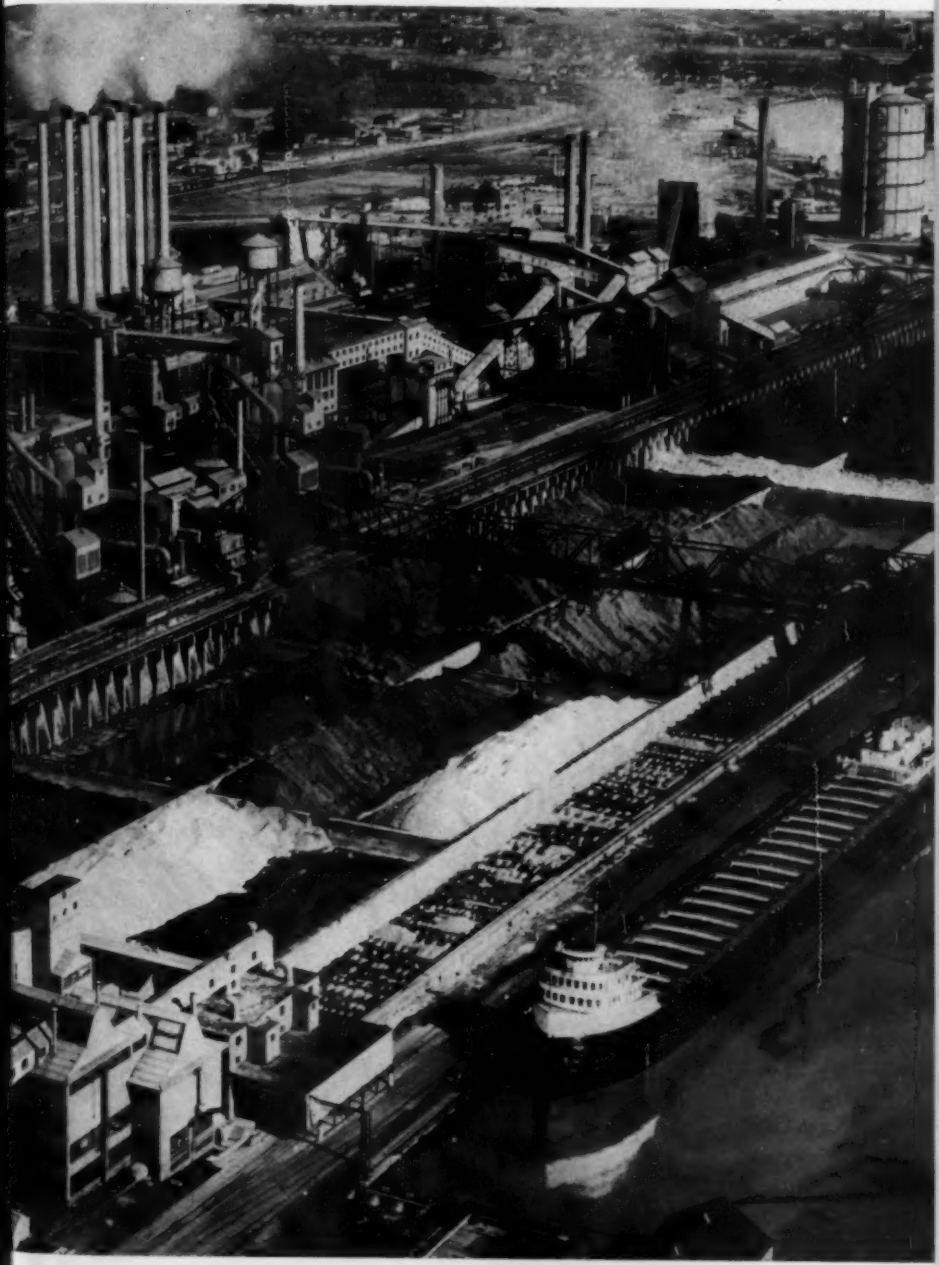
Vol. 30,

ACI News Letter

Vol. 30, No. 3 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1958



Industrial Detroit hosts ACI regional meeting . . . p. 3

Contributions to the

ACI JOURNAL

Technical pages of the ACI JOURNAL offer a medium for reporting what is new and interesting in concrete research, design, construction, maintenance, or manufacturing. ACI members and JOURNAL readers are urged to share their knowledge and experience by submitting papers and reports for publication in the ACI JOURNAL. Time and changing interests demand that new contributors with new ideas take their place beside the old.

●

The Technical Activities Committee selects the papers, committee reports, discussions, and other contributions for JOURNAL publication. A volunteer corps of experts in various segments of the field assists in technical appraisal of the manuscripts. Technically speaking, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

Further questions raised by TAC: will the contribution fit into a balanced publication schedule? Can it be published at reasonable expense to the Institute?

●

Contributors should furnish glossy prints or inked tracings of illustrations along with manuscripts submitted in triplicate for publication. Details of the form and physical arrangement of contributions are given in the 1957 *ACI Directory* p. 14. Separate copies of this "publications policy" are available free on request to the Secretary.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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ACI

News Letter Contents

September 1958

On the Cover—Just west of Detroit, the tall stacks of the Ford Motor Co. power plant, largest one in private industry, look down on the ore docks. Bridge cranes run back and forth over the storage bins filled with iron ore, limestone, and coal. The Ford Rouge plant, world's largest industrial concentration, covers 2 square miles of integrated production.

Photo courtesy Ford Motor Co.

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Detroit Regional Meeting Program Demonstrates

World-Wide Concrete Interest

October 27-29

Highlighted by the dedication of the new Institute headquarters building, the 11th ACI Regional Meeting promises an impressive program of technical papers to be presented by eminent speakers of national and international prominence.

The 3-day program, spiced with stimulating variety, includes technical sessions devoted to design, construction, the ACI Building Code, materials, and the ACI headquarters building. The sessions are geared to be of equal interest to the designer, builder, research man, and those in related fields of the concrete industry.

Emphasizing the world-wide scope of Institute activities, the speakers' roster for this meeting includes three European authorities.

A. M. Haas

A. M. Haas from Delft, Netherlands, will speak on "Relation of Form to Structural Design" at the design session, October 27. He is a staff member at the Institute of Technology, Delft, as professor of concrete and concrete construction. He graduated in 1921 and received his doctor's degree in 1948 from the Institute. His experience spans three continents. He worked until 1927 in the United States and then was engaged for 2 years in road and building construction in the Netherlands. The following 10 years found Professor Haas engaged in various phases of concrete construction in the Netherlands East Indies. Returning to the Netherlands he continued his concrete construction activity, joining the faculty at Delft in 1953. He is president of the Netherlands Concrete Association and of the Netherlands Committee for Concrete Research.

Georg Wastlund

Georg Wastlund, professor of bridge building and structural engineering at the Royal Institute of Technology, Stockholm, Sweden, and director of

WALTER H. PRICE (left) and JAMES D. PIPER were busy with candid cameras during their recent visit to the Soviet Union where they surveyed methods in the reinforced and prestressed concrete industry. Mr. Piper, vice-president for promotion, Portland Cement Association, Chicago, and Mr. Price, who heads the engineering laboratories of the Bureau of Reclamation at Denver, will report some of the highlights of their Russian tour for ACI regional meeting visitors, October 27.

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In the background are ACI members Ben C. Gerwick, Jr., president of Ben C. Gerwick Co., San Francisco, and Prof. T. Y. Lin, (right) University of California, Berkeley. Professor Lin headed the six-man American delegation.

the Swedish Cement and Concrete Research Institute, Stockholm, will speak at the building code session on "High Strength Steel and Cracking in Reinforced Concrete Members."

Professor Wastlund graduated as a civil engineer from the Royal Technical University, Stockholm, in 1928 and received his doctor's degree in 1934. Following graduation he was an assistant at the Structural Strength Laboratory of the university from 1929 to 1936. For the next 5 years he worked as a structural engineer at Skanska Cementgjuteriet, Malmo, Sweden; in 1941 he became professor of structural engineering and bridge building at the Royal Technical University. He is technical adviser to International Association for Bridge and Structural Engineering, a member of the bureau of the European Concrete Committee, and was president of the RILEM Symposium on Bond and Crack Formation in Reinforced Concrete in Stockholm in 1957.

Hubert Rüsch

Since publication of the tentative program, confirmation has been received that Hubert Rüsch, professor of reinforced concrete and bridge building and director of Laboratory for Testing Building Materials and Structures, Institute of Technology, Munich, Germany, will address the building code session on factors of safety involved in construction employing simple and composite bending.

In his present position since 1945, Dr. Rüsch has won great prominence through design and construction of outstanding reinforced concrete structures in Europe, England, and South America. He has played an active part in the development of shell structures, prestressed concrete work, and precast construction projects of Dyckerhoff and Widmann. Just prior to World War II the Franklin Institute of the State of Pennsylvania awarded him the Longstreth Medal in recognition of his work in shell design.

Building dedication

Following 2 days of intensive technical sessions, the ACI regional meeting will be topped off on October 29 with the official dedication of the new headquarters, cul-

minating years of planning and fund raising effort on the part of the ACI Building Committee. Tours of the building will be conducted from 9:30 to 12:00, followed by the dedication luncheon at the Statler Hilton Hotel with A. Allan Bates, chairman of the Building Committee presiding. ACI President McHenry will be joined on the speakers' platform at this ceremony by Robert F. Legget, director, Division of Building Research, National Research Council of Canada, Ottawa. During the past few years, Mr. Legget has visited building research institutes all over the world, as well as a large number of the major laboratories of the world working in cement and concrete. He is familiar with progress throughout the world in concrete construction and related subjects of interest.

In connection with the dedication ceremonies, the new library in the ACI headquarters building will be designated the Harvey Whipple Library in honor of the late ACI Secretary-Treasurer, who served the Institute continuously for 32 years. Mr. Whipple retired in February, 1952, and it was hoped that he would continue to serve for many years longer as editorial consultant, but his untimely death in September, 1952, ended his illustrious career.

Details of a special "Ladies Program" are being culminated to assure a thoroughly enjoyable schedule while husbands are engrossed in technical sessions and Institute affairs.

J. Gardner Martin, Portland Cement Association, Lansing, Mich., is general chairman of the regional meeting and Prof. Elihu Geer, University of Detroit, heads the planning group for the technical program which appears below.

MONDAY MORNING, OCTOBER 27

Design Session—
President Douglas McHenry, *Chairman*

Introductory Remarks—President McHenry

Russian Progress in Concrete Technology
J. D. Piper, Portland Cement Association, Chicago, and Walter H. Price, U. S. Bureau of Reclamation, Denver

Relation of Form to Structural Design
A. M. Haas, Institute of Technology, Delft, Netherlands

Strength of Concrete Under Combined Stresses
K. S. Pister and Boris Bresler, University of California, Berkeley

Analysis of Shear Test Data (work of ACI Committee 326)
I. M. Viest, AASHO Road Test, Ottawa, Ill.

Comparative Cost of Blast Resistant Design
Warren Yee, Smith, Hinchman, and Grylls, Detroit

MONDAY AFTERNOON, OCTOBER 27

Construction Session—
Ben Maibach, Jr., *Chairman*

Concrete Shell Roof Construction Problems
O. G. Sharrar, O. W. Burke Co., Detroit

Precast Sandwich Panels for Shopping Center
To be presented by a representative of Marietta Concrete Corp., Marietta, Ohio

Design Problems and Service Performance of Precast Concrete Panels
Victor Leabu, Giffels and Rossetti, Detroit

Concrete Formwork
Harry Ellsberg, Giffels and Rossetti, Detroit, Chairman ACI Committee 622

TUESDAY MORNING, OCTOBER 28

Building Code Session—
Raymond C. Reese, *Chairman*

High Strength Steel and Cracking in Reinforced Concrete Members
Georg Wastlund, Cement and Concrete Institute, Stockholm, Sweden

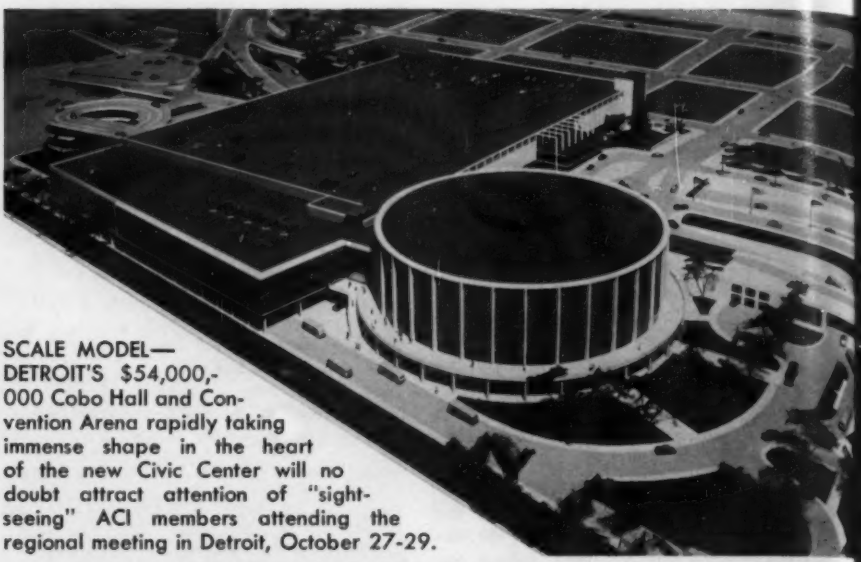
Safety, Simple and Composite Bending
Hubert Rüsch, Technische Hochschule, Munich, Germany

What's Coming in ACI Building Code Revisions
Raymond C. Reese, Consulting Engineer, Toledo, Ohio

Beam Creep and Deflection
George E. Large, Ohio State University, Columbus

New Grades of Reinforcing Steel and Status of Ultimate Strength Design
George Winter, Cornell University, Ithaca, N. Y.

Footings
Paul Rogers, Consulting Engineer, Chicago



SCALE MODEL—DETROIT'S $54,000,000 Cobo Hall and Convention Arena rapidly taking immense shape in the heart of the new Civic Center will no doubt attract attention of "sightseeing" ACI members attending the regional meeting in Detroit, October 27-29.

Cobo Hall, a giant rectangular building with 396,425 sq ft of floor space, is scheduled for completion May 1, 1960. The arena, with seating capacity for 14,000 plus 33 smaller rooms for meetings ranging in attendance from 80 to 3000, and cafeteria to seat 1500, is to be finished Dec. 1 of the same year.

The whole facility is located in the hub of Detroit's downtown hotel and shopping area. The roof of the exhibit area will serve as a parking lot and helicopter deck. Note that model pictured above shows exhibit portion spanning the Lodge Expressway. Completion of the convention-exhibit building will round out Detroit's great new Civic Center on the downtown riverfront. Designed by Giffels and Rosetti of Detroit; Darin and Armstrong, Inc., general contractor for the substructure; O. W. Burke Co., general contractor for the superstructure.

Flat Slabs and Flat Plates for Industrial Buildings
Alfred Zweig, Albert Kahn Associates, Detroit; Joseph DiStasio, Consulting Engineer, New York; J. Watts, Harley Ellington and Day, Detroit

TUESDAY AFTERNOON, OCTOBER 28

CONCURRENT SESSIONS

Materials Session—
Paul F. Rice, *Chairman*

Heavy Media Separation of Aggregate
David A. Killins, Killins Gravel Co., Ann Arbor, Mich.

Special Finishes and Colors in Precast Concrete Wall Panels
Will Saia, Precast Concrete Products Co., Royal Oak, Mich.

Efficiency of Stationary Tilting Mixers
John Winkworth, Winkworth Fuel and Supply Co., Detroit

Lightweight Slag Concrete
D. W. Lewis, National Slag Association, Washington, D. C.

ACI Building Session—
William Krell, *Chairman*

Precast Roof—Film and Commentary
William Krell, Portland Cement Association, Detroit

Architectural Concepts
Minoru Yamasaki, Yamasaki and Leinweber, Birmingham, Mich.

Design of ACI Building
C. S. Whitney, Ammann and Whitney, Milwaukee and New York

Construction Features
John Strang, Pulte-Strang, Inc., Ferndale, Mich.

WEDNESDAY MORNING, OCTOBER 29

DEDICATION OF ACI HEADQUARTERS

A. Allan Bates, *Chairman*

Tour of ACI Headquarters Building, 9:30 to 12:00

Dedication Luncheon—Statler Hilton Hotel, 12:30

Guest Speaker, Robert F. Legget, National Research Council of Canada, Ottawa

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President McHenry to attend Structural Engineers' conclave

ACI President Douglas McHenry will represent the Institute at the 50th anniversary celebration of the Institution of Structural Engineers, October 7-10, in London, England.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 213, Lightweight Aggregate and Lightweight Aggregate Concrete

J. A. Hanson
Portland Cement Association
Chicago, Ill.

Committee 616, Coatings for Concrete

C. A. Sentel
National Bureau of Standards
Washington, D. C.

Committee 716, High Pressure Steam Curing

William H. Kuenning
Portland Cement Association
Chicago, Ill.

Los Angeles committee announces plans for '59 ACI convention

Sam Hobbs, local chairman for the 55th annual convention of the American Concrete Institute to be held in Los Angeles, Feb. 23-26, 1959, has announced preliminary plans and committee appointments for convention activities. Convention headquarters will be at the Statler Hilton Hotel.

The general committee to correlate all local planning will include John McNerney, Portland Cement Association; Byron P. Weintz, Consolidated Rock Products; William F. Norton, Ceco Steel Products Corp.; C. Taylor Test, Riverside Cement Co.; Glenn C. Thomas, Thomas Concrete Accessory Co.; Lewis K. Osborn of Kistner, Wright and Wright; and Ross Adams of the Portland Cement Association.

C. Taylor Test will serve as chairman of the entertainment committee and will announce later committee personnel in charge of the ladies' program and other social events planned for the convention.

Glenn C. Thomas will head up the exhibits committee which is planning a large-scale, high quality exhibit of services and specialties in the concrete industry.

Lewis K. Osborn will take over the hotel arrangements committee.

The first two convention days will be devoted entirely to technical committee meetings and a session on ACI standards.

The formal technical program will begin on Wednesday, February 25, with two sessions

Continued on p. 10

ATTENTION EXHIBITORS

The local committee for the 55th ACI Annual Convention in Los Angeles, Feb. 23-26, 1959, will sponsor an exhibit in conjunction with the convention.

Firms desiring to exhibit are requested to contact:

Glen Thomas
c/o Southern California Chapter
American Concrete Institute
Room 1058, 742 S. Hill St.
Los Angeles 14, Calif.

new church design shows versatility of PRECAST, PRESTRESSED CONCRETE

Working together, two cranes position prestressed slabs. 20 slabs are 5½" thick, 4' wide and 38½' long; 8 slabs are 4" x 4' x 38½'; there are 10 shorter slabs 4" x 4' x 30'.

On the interior, prestressed slabs were sprayed with an acoustical plaster.

• The St. Ambrose Episcopal Church in West Fort Lauderdale, Fla. is another example of the almost unlimited possibilities of modern precast, prestressed concrete construction.

Just 38 prestressed slabs were required for this unique structure. Cast with plates imbedded at the ends and along the sides, slabs were fitted into slots in the footings, hoisted into place and welded together. The joints were coated with roofing material. That's all there was to it.

By eliminating conventional walls and roof, this technique resulted in substantial savings to the owner. The appearance of the completed job speaks for itself.

The prestressed slabs were made with Lehigh Early Strength Cement at the R. H. Wright & Son, Deerfield plant. Its use helps them save time and money in producing a variety of top quality concrete units.

ARCHITECT: Johnson and McAlpine, Ft. Lauderdale, Fla.
CONTRACTOR: Quick Quality Construction, Inc., Ft. Lauderdale, Fla.
PRECAST, PRESTRESSED CONCRETE UNITS MANUFACTURED BY: R. H. Wright & Son, Ft. Lauderdale, Fla.

Ample daylight flows into the church through the 5 open spaces on each side.

LEHIGH PORTLAND CEMENT COMPANY
Allentown, Pa.

Positions and Projects

Paul F. Rice joins Concrete Reinforcing Steel Institute

Paul F. Rice, formerly ACI technical director, recently joined the staff of the Concrete Reinforcing Steel Institute, Chicago, as assistant to Harry C. Delzell, managing director. He will engage in field and office work in the promotion and technical aspects of the use of reinforcing steel in concrete construction.

Mr. Rice received his BS at North Dakota State College in 1941 and his MS at Massachusetts Institute of Technology in 1947.

During World War II he served 42 months in a U. S. Army aviation engineer battalion with responsibilities for paving, building construction, labor and supply procurement, and earthmoving. He was discharged with the rank of captain in 1946.

Mr. Rice's professional experience includes structural detailing and shop inspection of fabricated steel, field inspection and laboratory testing of concrete, structural designing, estimating, assistant instructor in drafting, and 5 years as a structural field engineer for the Portland Cement Association. He joined the ACI staff as technical director in 1954.

Active in a number of civic and professional groups, Mr. Rice was a director of the Concrete Improvement Board of Metropolitan Detroit. He is also a member of the Detroit chapter of Michigan Society of Professional Engineers, American Society of Civil Engineers, and Construction Specification Institute. He has been active in the Engineering Society of Detroit; MIT Alumni Association of Detroit; and the Society of American Military Engineers, Detroit Post.

Past president Moreell retires

ACI past president Ben Moreell, chairman of the board of Jones and Laughlin Steel Corp., will retire as of Sept. 30, 1958. He will continue as director and a member of the executive committee of the board of directors.

Retiring from active service in the Navy in 1946, Admiral Moreell joined Jones and Laughlin channeling his energies into directing a $500,000,000 expansion program. He served Jones and Laughlin as chairman of the board, president and chief executive officer from 1947 to 1952; chairman of the board and chief executive officer until October 1957, and as chairman of the board since the latter date.

During his nearly 30 years of naval service, Admiral Moreell rose from the ranks to become the only non-Annapolis graduate ever to attain the permanent rank of admiral. In recognition of his accomplishments for the Navy, Admiral Moreell has been awarded the Distinguished Service Medal with Gold Star; the Legion of Merit; the Order of the British Empire, Military Branch, Rank of Commander; and the Medal of Honor and Merit from the Republic of Haiti.

Admiral Moreell became a member of the Institute in 1930, a member of the board in February 1935, and president in 1941. He served 12 years on the Institute's board of direction.

Historical Book Donated to ACI Library

B. H. Armiger, Detroit ACI member, has kindly donated to the Harvey Whipple Library at ACI headquarters a copy of L. J. Mensch's book, *The Reinforced Concrete Pocket Book*, copyright 1909.

The headquarters staff would be pleased to learn from other members of books of similar historical interest which they might wish to contribute to the library.

Benn establishes consulting firm

Philippe A. Benn announces the establishment of his own practice under the name of Philippe A. Benn and Associates, consulting engineers, with headquarters in Montreal, Que., Canada.

Mr. Benn is also executive vice-president of Integrated Consultants, Ltd., consulting process and mechanical engineers, Montreal.

'59 ACI convention

Continued from p. 7

running concurrently through Thursday noon, to be followed by the final research session on Thursday afternoon. California authors and speakers have been given a prominent part in the convention program.

Entertainment and hospitality features being arranged for all convention attendants will include a Wednesday evening social hour and a trip to Disneyland on Friday, February 27. Several events are being arranged for the ladies, including a get-acquainted brunch early in the week and a lunch and special program at the Beverly Hilton Hotel on Thursday.

As an added feature of the convention, the exhibits committee will hold a student competition with awards for models and other presentations depicting concrete designs and applications.

St. Lawrence University receives Steinman grant

Dr. David B. Steinman, internationally eminent bridge engineer, has made a grant of $10,000 to St. Lawrence University to establish Holton D. Robinson scholarships for student aid, in memory of his late partner, Dr. Robinson, who was an alumnus of St. Lawrence.

The grant specifies that each student recipient of a scholarship be asked to assume a moral obligation to make repayment to the university when he can at any future time so as to help make the scholarship fund self-perpetuating. The underlying thought is to inculcate the spirit of giving to help successive generations of students. Dr. Steinman calls this "a moral chain-reaction" of obligation and opportunity.

To date, Dr. Steinman has made grants to more than 30 educational institutions, in an aggregate amount exceeding $400,000.

Eskenazi opens consulting office

Beno A. Eskenazi, consulting structural engineer, has opened his own office in Ville St. Laurent, Que., Canada. Mr. Eskenazi was formerly chief engineer at Henry Jansen, Ltd., in Montreal.

Cleveland Engineering Society plans construction conference

Plans are being completed for the Cleveland Engineering Society's 1958 Construction Conference scheduled for Nov. 12 at the new Cleveland Engineering and Scientific Center in Cleveland.

Theme for the all-day conference is "Construction Countdown" and will feature nationally known speakers discussing current problems in the fields of construction, architecture, and engineering.

Programs will be sent to interested persons who write to the society at 3100 Chester Ave., Cleveland 14, Ohio.

Basalt establishes consulting service

The establishment of a new consulting service on the manufacture of plant cast, prestressed concrete products has been announced by A. G. Streblow, president of Basalt Rock Co., Napa, Calif.

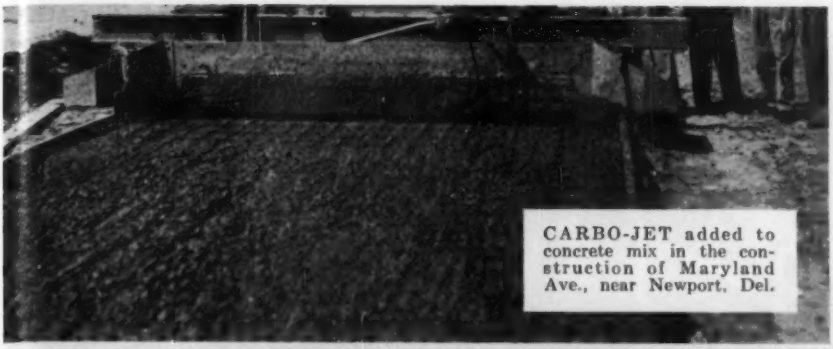
This service is designed to aid concrete manufacturers in entering this field. Consultants will cover every phase of the business, including such basic problems as plant design to satisfy marketing designs of particular areas, and selection of products and shapes to be manufactured.

Proposed changes in pump and mixer standards studied

Proposed changes in contractors' pump standards and concrete mixer standards were discussed at the summer meetings of the Contractors Pump Bureau and the Mixer Manufacturers Bureau at Chatham, Mass.

Further consideration of the proposed changes in the standards will be given at the annual meetings of the bureaus in Chicago in December. Actions of the two bureaus on these changes will be subject to approval by the Associated General Contractors of America.

The mixer bureau also reviewed proposed changes in its standards and a special committee was appointed to study the portion related to the 3½ S type mixer. The bureau recently extended its standards to include plaster and mortar mixers, and is now considering the feasibility of including concrete paving machinery.



CARBO-JET added to concrete mix in the construction of Maryland Ave., near Newport, Del.

The sixth major job on which CARBO-JET has been used by the Delaware State Highway Dept.

CARBO-JET: The quick, efficient method to darken concrete

CARBO-JET reduces glare, improves appearance of roads, walks, curbs, center islands, patios and driveways.

CARBO-JET is a specially formulated carbon black, dispersed in liquid form. It is by far the easiest, and most economical way to introduce gray-to-black shades in mortar and plain or air-entrained concrete. It disperses faster than ordinary cement colors. Takes much less mixing time. There is no streaking, no color variation. CARBO-JET does not affect the air content of air-entrained concrete.

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Successful usage in millions of yards of concrete has proved you save money and get better results at every stage when you add *liquid* J-M Placewel to your mix. You get stronger, more durable concrete—at lower construction and maintenance costs.

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National Construction Industry Conference scheduled in December

Creative trends in urban building will be the theme of the fourth annual National Construction Industry Conference to be held in the Sherman Hotel, Chicago, on Dec. 10 and 11.

Sponsors of the conference are Armour Research Foundation of Illinois Institute of Technology, American Institute of Architects, American Society of Civil Engineers, Associated General Contractors of America, and the Building Research Institute.

The program will include some 16 papers dealing with new structural concepts, elements, and techniques as well as social, economic, political, and other forces influencing modern construction.

Inquiries concerning the conference should be addressed to conference chairman, R. T. Mijanovich, Armour Research Foundation, 10 W. 35th St., Chicago 16, Ill.

Munger joins Norwich University faculty

Elmer L. Munger, formerly of the department of theoretical and applied mechanics, Iowa State College, Ames, has accepted appointment as professor of civil engineering and head of the department at Norwich University, Northfield, Vt.

Stone and Webster Engineering forms Netherlands subsidiary

Stone and Webster Engineering Corp., Boston and New York, recently announced expansion of its foreign operations in Europe with the formation of a Netherlands subsidiary and a further integration of its British, French, and Australian subsidiaries. Both the British and French subsidiaries are active in the field of engineering and construction largely in the petroleum and petrochemical industries throughout western Europe.

Driscoll promoted

George C. Driscoll, instructor in civil engineering for the past 5 years at Lehigh University, has been named assistant professor effective next academic year. Joining the staff at Lehigh as a graduate assistant, he has taught elementary mechanics and a graduate course in research methods. In his capacity as assistant engineer of tests in Fritz Engineering Laboratory, Professor Driscoll supervises industrial testing work of the laboratory and the operation of large-scale fatigue testing facilities.

LOOKING AHEAD

Oct. 13-17, 1958—American Society of Civil Engineers, Annual Convention, Statler Hotel, New York, N. Y.

Oct. 20-24, 1958—National Safety Congress and Exposition, Conrad Hilton and other hotels, Chicago, Ill.

Oct. 23-25, 1958—National Society of Professional Engineers, Fall Meeting, St. Francis Hotel, San Francisco, Calif.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hilton Hotel, Detroit, Mich.

Nov. 12, 1958—Cleveland Engineering Society, 1958 Construction Conference, Cleveland Engineering and Scientific Center, Cleveland, Ohio

Nov. 18-20, 1958—American Standards Association, Ninth National Conference on Standards, Hotel Roosevelt, New York, N. Y.

Dec. 7-10, 1958—American Institute of Chemical Engineers, Annual Meeting, Netherland Plaza Hotel, Cincinnati, Ohio

Dec. 10-11, 1958—National Construction Industry Conference, Fourth Annual Meeting, Sherman Hotel, Chicago, Ill.

Feb. 23-27, 1959—American Concrete Institute, 55th Annual Convention, Statler Hilton Hotel, Los Angeles, Calif.

Orlando William Irwin

Orlando William Irwin died suddenly July 26 in Bangor, Me. At the time of his death Mr. Irwin was president of the Rail Steel Bar Association.

Mr. Irwin was educated at Wooster College and Case Institute of Applied Science. His earliest experience was as a civil engineer with the Lake Shore Railroad. In 1911 he joined Truscon Steel Co. remaining there until 1935. He then became associated with Carnegie-Illinois Steel Corp. until 1946 when he left to become president of the Rail Steel Bar Association.

His principal interest has been structural engineering and the advancement of reinforced concrete. He was a charter member, holding various offices in the Concrete Reinforcing Steel Institute. He helped organize and was a president of the Steel Joist Institute, and also was active member and president of the reinforcing materials fabricating industry section of the NRA Code Authority, was chairman of the Concrete Bar Producers Committee for Defense Construction during the war, and an organizer and chairman of the American Iron and Steel Institute committee on reinforced concrete research.

Mr. Irwin had been a member of ACI since 1916 and was a life member of the American Society of Civil Engineers. At the time of his death Mr. Irwin was a member of ACI Committee 317, Reinforced Concrete Design Handbook.

Norman M. Stineman

Norman M. Stineman, an ACI member of 28 years standing, died recently in Portland, Ore.

Mr. Stineman received his civil engineering degree in 1911 from Ohio Northern University. The 6 years following graduation were spent in the engineering department of the Chicago, Milwaukee and St. Paul Railway at Chicago, designing grain storage terminals, steel and concrete bridges, viaducts, and railway passenger and freight stations. During this period he was awarded the Octave Chanute bronze medal of the Western Society of Engineers for developing original and workable formulas for several types of engineering structures.

In 1917 he resigned his position as chief engineer of LaSalle Engineering Co., Chicago, to enter military service in World War I. While stationed in France he was engineer officer in charge of construction of the car repair yard at Nevers.

In 1919, after a few months as chief draftsman of the Bureau of Bridges of the Indiana State Highway Department, Mr. Stineman joined the Portland Cement Association. For the next 10 years at PCA, he was in charge of special work on revision and preparation of city building regulations; he also specialized in fireproof construction, bridge and dam design, structural effects of hurricanes and earthquakes. During 1929 he assisted in drafting a new building code for New York City as a member of the New York Building Code Subcommittee on Reinforced Concrete.

Mr. Stineman became editor of *Concrete* in 1930 and in this capacity spent the next 13 years promoting the design and development of reinforced concrete. He resigned his editorship in 1943 to take up government work in Washington, D. C., with the Far Eastern Command during World War II. He continued government work as a structural engineer in Chicago after the war and was working with the Cook County Highway Department on the new freeways when he retired in 1955.

Robert R. Shoemaker

Robert R. Shoemaker, former chief harbor engineer of the Port of Long Beach and one of the harbor's builders, died recently at his home in Long Beach after an extended illness.

Following graduation from Purdue University in 1920, Mr. Shoemaker became an employee of the Los Angeles Port and advanced through the ratings of assistant structural engineer, structural engineer in charge of design, specifications and contracts engineer, and civil engineer and chief draftsman.

In 1940, Mr. Shoemaker was appointed chief harbor engineer at the Port of Long Beach and remained in that capacity until his death. Within the period of his service he saw the harbor grow in size and prominence among the ports of the world and personally played a major part in that growth.

Mr. Shoemaker's professional activities were many, including the presidency of three major engineering societies in California. He held memberships in a number of professional societies and had been a member of ACI since 1945.

George Dunton Youngclaus

George Dunton Youngclaus, widely known in engineering and concrete construction circles on the West Coast, died on July 25 at the age of 37 in Whittier, Calif.

In the few years of his engineering career in the Los Angeles area he had gained an enviable reputation for ability and integrity. Specializing for the past 2 years in the field of prestressed concrete lift-slab construction, he had become vice-president of Western Concrete Structures Co., Inc., and had played an important part in the building of many modern and outstanding structures in which the company was engaged.

His college career at the University of Colorado was interrupted by his entering the Army Air Force in 1943. However, following the war he returned to the university, graduating in 1947. For 2 years he was employed in an architectural office in Denver, moving to Los Angeles in 1949.

Mr. Youngclaus then became a field engineer for the Portland Cement Association, Los Angeles district office, specializing in structural work. For the next 7 years he extended and developed his knowledge and experience in new structural types and was in demand for advisory and lecture work on prestressed concrete, thin shell, and lift-slab construction, and on other concrete structural problems. He left PCA in 1956 to join Western Concrete Structures.

He became a member of the American Concrete Institute in 1948. He was an associate member of both the American Society of Civil Engineers and the Structural Engineers Association of Southern California. Through his firm, he had close association with the Prestressed Concrete Institute and as president of the Fire Prevention Research Institute, he engaged in the planning of fire tests to develop information on prestressed concrete.

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Concrete sets faster, tests stronger, requires less cold weather protection.

Columbia Calcium Chloride

KEEPS HOT SCHEDULES FROM COOLING OFF

Hot job schedules and falling temperatures used to turn contractors' hair snow-white overnight. Barbers and wives report this isn't nearly as great a problem today. News of Columbia Calcium Chloride has gotten around.

Even at 40° and below (not so far away now) concrete averages *three times faster* on both initial and final set when Columbia Calcium Chloride treated mix is used. There's something about this unique chemical agent that just warms up to chilly weather.

You meet strength specs days ahead, pull forms for other jobs, get finishers on and off the site without profit-killing overtime. *Right now* is the perfect time to analyze your batching procedures, or to talk to your ready mix suppliers.

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San Francisco. IN CANADA: Standard Chemical Limited

Who's Who This Month

ACI Committee 613

"Proposed Recommended Practice for Selecting Proportions for Structural Lightweight Concrete," submitted by the Subcommittee on Proportioning Lightweight Aggregate Concrete, ACI Committee 613, appears on p. 305.

J. J. Shideler, Portland Cement Association, Chicago, is chairman of the subcommittee with Robert K. Duey, consulting engineer, Gaithersburg, Md.; Frank Erskine, Expanded Shale, Clay and Slate Institute, Washington, D. C.; Richard J. Frazier, Anchor Concrete Products, Inc., Buffalo, N. Y.; and Fred Hubbard, Standard Slage Co., Youngstown, Ohio, serving as committee members.

The late Henry L. Kennedy, formerly associated with the Dewey and Almy Chemical Co., Cambridge, Mass.; Thomas B. Kennedy, Waterways Experiment Station, Corps of Engineers, Jackson, Miss.; George H. Nelson, Law Engineering Testing Co., Atlanta, Ga.; and Carl M. Rollins, Basalt Rock Co., Inc., Napa, Calif., also served as members of the subcommittee preparing this report.

Balance of the committee was comprised of H. C. Ross, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada; L. A. Thorssen, Herculite Light Weight Aggregates of Canada, Ltd., Calgary, Alta., Canada; Stanton Walker, National Ready Mixed Concrete Association, Washington, D. C.; Cedric Willson, Texas Industries, Inc., Fort Worth, Tex.; and Paul M. Woodworth, Waylite Co., Chicago, Ill.

W. A. Cordon, Utah State Agricultural College, Logan, is chairman of ACI Committee 613, Recommended Practice for Proportioning Concrete Mixes, of which this subcommittee is a unit.

Subcommittee VIII, ACI Committee 325

Report of Subcommittee VIII, ACI Committee 325, on "Design of Concrete Overlays for Pavements" is on p. 315 of this issue.

Leo M. Arms, Portland Cement Association, Chicago, is chairman of the subcommittee with Henry A. Aaron, Wire Reinforcement Institute, Washington, D. C., and L. A. Palmer, Bureau of Yards and Docks, Department of the Navy, Washington, D. C., serving as committee members.

E. A. Finney, Michigan State Highway Department, East Lansing, is chairman of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports, which now has eight subcommittees functioning.

Boris Bresler and K. S. Pister

"Strength of Concrete Under Combined Stresses" on p. 321 was coauthored by Boris Bresler and K. S. Pister.

Boris Bresler is a faculty member at the University of California, Berkeley, in the capacity of professor of civil engineering and assistant dean in charge of graduate study, department of engineering. Professor Bresler graduated from the University of California in 1941, receiving his MS in aeronautical engineering from the California Institute of Technology in 1946.

Following graduation in 1941, he engaged in engineering work with Kaiser Shipyards, Inc., Richmond, Calif., and Consolidated Aircraft Corp., San Diego, through 1945. He then returned to the California Institute of Technology, Pasadena, as a graduate fellow. In 1946 he joined the faculty at the University of California.

Professor Bresler is a member of ACI-ASCE Committee 326, Shear and Diagonal Tension, and an alternate member of the Reinforced Concrete Research Council.

K. S. Pister received a BS in civil engineering from the University of California, served in the U. S. Navy Civil Engineer Corps, and in 1946 returned to the University of California for graduate work. Receiving his MS in 1948, he served as lecturer and instructor in theoretical and applied mechanics at the University of Illinois, while

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pursuing graduate work in that department. He received his PhD in 1952.

Dr. Pister then rejoined the faculty of the University of California where he has specialized in structural mechanics.

Edward K. Rice

"Economic Factors in Prestressed Lift-Slab Construction" on p. 347 was submitted by Edward K. Rice, partner in the consulting engineering firm of T. Y. Lin and Associates, Van Nuys, Calif.

Mr. Rice received both a BS and MS from the University of California at Berkeley and was a lecturer in engineering at the University of California in Los Angeles for several years.

Until recently he was partner and chief engineer of Western Concrete Structures Co. In this capacity he was responsible for the development of many new techniques in prestressed concrete lift-slab construction.

Mr. Rice has served on a number of technical committees including the task committee of ACI Committee 323, Prestressed Concrete Institute committee on fire testing, PCI Double Tee Standardization Committee, and is chairman of the PCI Diaphragm and End Block Committee for Prestressed Concrete Bridges.

J. E. Backstrom, R. W. Burrows, R. C. Mielenz, and V. E. Wolkodoff

Messrs. Backstrom, Burrows, Mielenz, and Wolkodoff now present the third part of the four-part paper on "Origin, Evolution, and Effects of the Air Void System in Concrete" on p. 359.

Part 3 is entitled "Some Factors Influencing Size and Spacing of Air Voids in Concrete and Their Effect on Freezing and Thawing Durability." Part 1 was published in the July 1958 ACI JOURNAL; Part 2 appeared in the August issue.

Biographical sketches on these authors appeared in the July and August issues.

Harold Samelson and Abba Tor

"Stresses in Reinforced Concrete Sections Subject to Transient Temperature Gradients" on p. 377 is the joint contribution of Harold Samelson and Abba Tor.

Mr. Samelson received his BS in civil engineering in 1944 from the College of the City of New York. Following graduation he worked for the Armour Research Foundation in the field of applied mechanics and did postgraduate work in that field at the Illinois Institute of Technology.

After leaving the foundation, Mr. Samelson worked for Sargent and Lundy, Chicago, and Sanderson and Porter, New York, designing heavy reinforced concrete power plant structures.

In 1950 he joined Ammann and Whitney where he is currently employed as assistant structural engineer. His experience has been mainly in bridges and special structures.

Abba Tor graduated from the Israel Institute of Technology, Haifa, in 1946; he received his MS in 1953 from Columbia University.

Between 1946 and 1950 he was employed as structural designer and field supervisor with engineering and architectural firms in Israel. From 1950-53 he worked as structural engineer for James Ruderman, consulting engineer, New York, where he was engaged in design of multistory buildings and auditoriums.

In 1953, Mr. Tor became a partner in a consulting engineering firm in Tel Aviv, Israel, specializing in the design of reinforced concrete structures.

Returning to the United States in 1957 he joined the Ammann and Whitney staff in the special structures department, where he is currently engaged in the design of large span reinforced concrete structures. Mr. Tor has been an ACI member since 1950 and is an associate member of ASCE.

J. A. Hanson

J. A. Hanson, development engineer in the Products and Applications Section, Portland Cement Association, Chicago, is author of the article, "Shear Strength of Lightweight Reinforced Concrete Beams," p. 387.

Mr. Hanson graduated from the University of Colorado Engineering School in 1939. After approximately a year with the International Boundary Commission on flood control projects in Texas, he returned to Denver to work for the U. S. Bureau of Reclamation. For the following 6 years he

was attached to the dams design division of the bureau. In this capacity he was particularly concerned with the study of the structural behavior of large concrete dams as determined by field measurements of strain, stress, deflection, and temperature.

In 1947, Mr. Hanson was transferred to the structural research section of the reclamation laboratories. In this section his work involved study of triaxial strength of concrete and rock materials, creep properties of these materials, and laboratory investigation of special design problems.

Leaving the Bureau in 1953, he spent 2 years in the Hawaiian Islands as head of the engineering department of Terminal Steel Co. In 1955 he returned to the mainland and joined the staff of the Portland Cement Association.

Mr. Hanson has been a member of ACI since 1947 and is a member of ACI Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete.

Errata

The following correction should be made in "Tentative Recommendations for Prestressed Concrete," reported by ACI-ASCE Committee 323, which appeared in the January, 1958, JOURNAL.

p. 560—Section 209.2.3(b), the first b in the equation should be b'.

* * *

The following correction should be made in "Origin, Evolution, and Effects of the Air Void System in Concrete. Part 2—Influence of Type and Amount of Air-Entraining Agent," by James E. Backstrom et al., which appeared in the August, 1958, JOURNAL.

p. 271—First line of type above Fig. 6 should read in part, "Fig. 6 shows the correlation between $\overline{L}$ and ..."

* * *

The following correction should be made in "Recommended Practice for Design of Concrete Pavements (ACI 325-58)," reported by ACI Committee 325 in the July, 1958, JOURNAL.

p. 39—Eq. (13) should read:

$$L_T = \frac{2 f_s A_s'}{u Q} + 3 \text{ in.}$$

Prestressed Concrete Institute to convene in Chicago

The annual meeting and convention of the Prestressed Concrete Institute will be held September 21-25 in Chicago, with headquarters at the Edgewater Beach Hotel.

Program sessions will cover marketing and sales of prestressed concrete; handling, transportation, and erection techniques; grouting; draping strands; specifications and inspection; long span structures. There will also be an exhibit of material and equipment relating to prestressing.

Tigrak to practice in United States

Dr. M. Fuat Tigrak, formerly of Ankara, Turkey, is now an associate in the firm of Clark, Daily and Dietz, consulting engineers, Urabna, Ill. He has been an ACI member since 1944.

National Colorcrete Advisory Board formed

To serve the concrete products industry and Colorcretors throughout the nation, the National Colorcrete Advisory Board has been established, according to a recent announcement by Russel C. Reeve, executive secretary.

The group will assist in researching new developments and establishing quality control standards for the Colorcrete industry. Headquarters are in Chicago.

Livesay named manager of Spanblok division

Harvey R. Livesay, Jr., has been named manager of the Spanblok division of Prestressing, Inc., San Antonio, according to a recent announcement by the company president, James J. Mennis.

Mr. Livesay graduated from the U. S. Military Academy and served 6 years in the Corps of Engineers. Later he received his MS in civil engineering from the University of Texas and was recently a structural engineer with Prestressing Research and Development, Inc., San Antonio.

He is a member of ACI as well as the American Society of Civil Engineers and the National Society of Professional Engineers.

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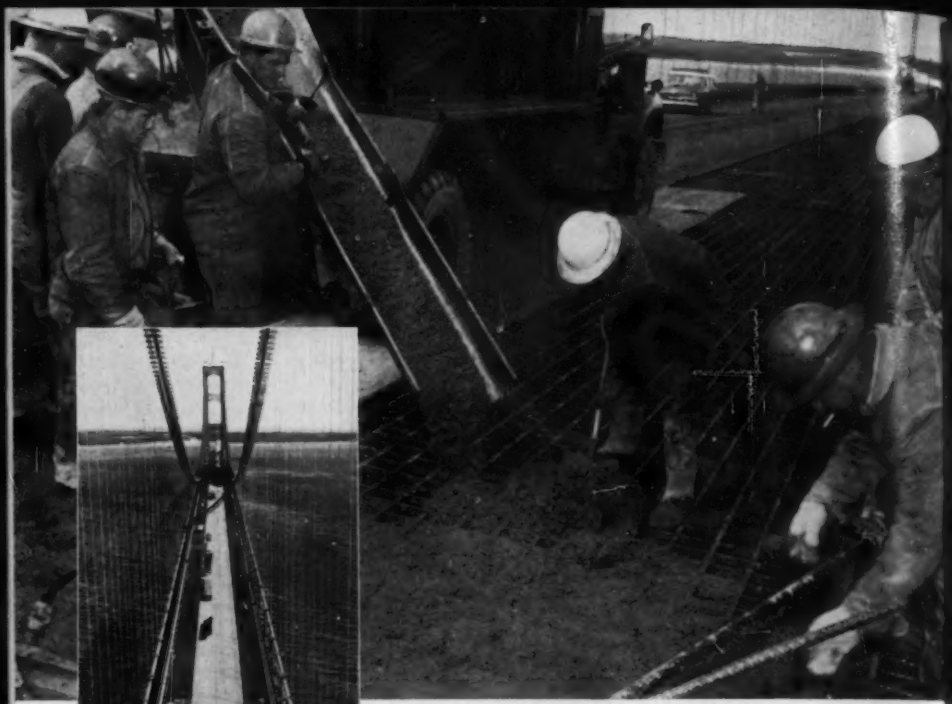

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Honor Roll

January 1—August 31, 1958

Eighty-four Institute members appear on this month's Honor Roll, with the top three "member-getters" in the same order as last month. Each month the Honor Roll lists individuals who have invited their associates to join the Institute.

Blas Lamberti	14
Joseph J. Waddell	11
Phil M. Ferguson	8
James A. McCarthy	7
L. M. Legatski	6
Ernst Maag	6
H. C. Pfannkuche	6
Jerome M. Raphael	6
J. P. Thompson	6
Samuel Hobbs	5
C. P. Siess	5
Ernest L. Spencer	5
Moises Bendahan	4
Celso A. Carbonell	4
P. E. Ellen	4
N. J. Everard	4
Carl H. Koontz	4
Frank E. Legg, Jr.	4
Douglas McHenry	4
John B. Porter	4
Clarence A. Walkwitz	4
Jaime de las Casas	3
Carl E. Ekberg, Jr.	3
Aleck S. Evans	3
Richard W. Gunn	3
Martin J. Gutzwiller	3
Paul A. Hansen	3
Robert B. Harris	3
E. L. Howard	3
Kenneth M. Huber	3
E. Richard Kummerle	3
Daniel S. Ling	3
George A. Mansfield	3
H. P. Mittet	3
Walter H. Price	3
E. Schmid	3
C. H. Scholer	3
Howard Simpson	3
O. C. Struthers	3
Byron P. Weintz	3
William M. Avery	2
Robert B. Banning	2
Nicandro Jose Barboza	2
Luther E. Bell	2
O. R. Bell	2
Massimiliano R. Berretti	2
Carlos D. Bullock	2
Vincent R. Cartelli	2
Miles N. Clair	2
W. L. Collier	2
W. S. Cottingham	2
Lancelot A. Fekete	2
Louis A. Gottheil	2
Einar Block Hansen	2
Arch Hull	2
Thomas C. Kavanagh	2
F. R. Khan	2
R. G. King	2
Carl O. Knop	2
E. Vernon Konkel	2
George Kurio	2
James R. Libby	2
Ciceron Hiedra Lopez	2
William McGuire	2
Bryant Mather	2
Frank B. May	2
James A. Murray	2
Wendell H. Nedderman	2
Abdur Rahman S. Rasul	2
Clarence Rawhauser	2
Herbert Rusk	2
Francesco Sardella	2
George Shervington	2
Otis D. Small	2
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Ferruh Taskin	2
T. W. Thomas	2

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New Members

The Board of Direction approved 69 Individual applications, 4 Corporations, 14 Juniors, and 6 Students, making a total of 93 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership on Aug. 1, 1958, was 9561.

Individual

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ARNI, HOWARD T., Washington, D. C. (Matls. Engrs., NBS)
AVERY, HARRY F., Denver, Colo. (Matls. Engr., USBR)
BARBER, JAMES, Victoria, B. C., Canada (Dir., Engrg. & Insp., G. S. Eldridge & Co., Ltd.)
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BROWN, CARROLL L., Tacoma, Wash. (Partner, Brown & Sleavin, Prof. Engrs.)
CAMPBELL, DR. GORDON D., Ottawa, Ont., Canada (Dir. of Tech. Services, The Library, Canadian Good Roads Association)
CHEN, T. Y., Hanover, N. H. (Assoc. Prof. of Civil Engrg., Thayer School of Engrg., Dartmouth College)
CHOW, EDWARD, Vancouver, B. C., Canada (Partner & Chf. Engr., Dunsmuir Constr., Ltd.)
DAVIS, LESLIE R., Chicago, Ill. (Tech. Service Engr., Medusa Portland Cement Co.)
DONNER, NOAH J., Calverton, N. Y. (Retired P. E.)
DURFEE, RAYMOND F., Lansing, Mich. (Asst. to Road Constr. Engr., Mich. State Hwy. Dept.)
EASTMAN, JULIUS D., S. Ozone Park, N. Y. (Struct. Engr., (Foundations) N. Y. Dist. Corps of Engrs.)
ENSLOW, JOHN T., Park Forest, Ill. (Arch., Housing & Cement Products Bur., PCA)
FEIBUSCH, HANS A., San Francisco, Calif. (Engr., Ben C. Gerwick, Inc.)
GENDRON, MICHEL, Quebec City, Canada (Prof. of R/C, Laval University, Faculty of Sciences)
HALL, IVAN, Ephrata, Wash. (Struct. Engr., USBR)
HALL, TREVOR W., Clawson, Mich. (Vice-Pres. & Gen. Mgr., Panel Engrg. Corp.)
HAYNES, M. A., Niagara Falls, Ont., Canada (Sr. Draftsman, H. G. Acres, Ltd.)
HENRY, V. L., JR., Falmouth, Mass. (Constr. Supt., Blount Bros. Constr. Co.)
HEPPENSTALL, C. W., Delray Beach, Fla. (Inventor)
HOBBS, M. H., Silver Spring, Md. (Chf. Struct. Engr., Mills, Petticord & Mills, Archs. & Engrs.)
HOFELDT, HENRY, New York, N. Y. (Partner, Weiskopf & Pickworth)
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HOLZMAN, LEWIS H., Framingham, Mass. (Struct. Design, Fay, Spafford & Thorndike, Inc.)
JETT, GEORGE B., Memphis, Tenn. (Consulting, Struct.)
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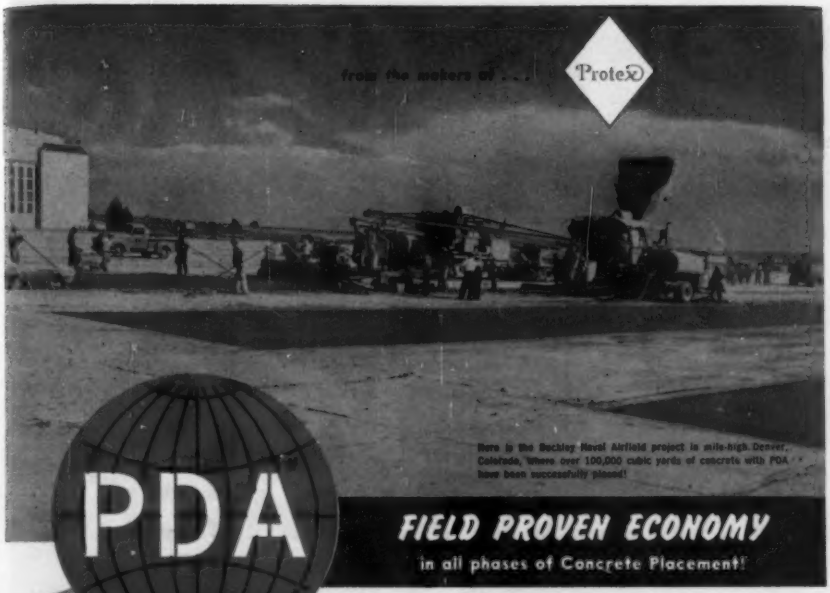

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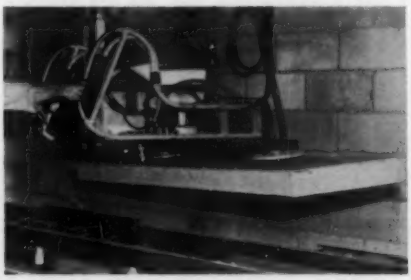
Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Flexible post-tension casing

U-100WP Econoflex identifies a new plain steel, flexible post-tension casing with superior crushing strength according to the manufacturer. It is fabricated from 0.012-in. strip steel with a four-wall metal thickness and fully interlocking, spiral tight seam to support the flexible tube against impact loads. Sizes range from 1⅜ to 2½ in. inside diameter; grouting inlet connections are available, factory attached, or in 24-in. flexible casing length sections for field splicing. —Universal Metal Hose Co., 2133 South Kedzie Ave., Chicago 23, Ill.

Vacuum lifting unit for concrete products

Four Vac-U-Lift units with push-button controls and built-in hand grips for handling maneuverability have been installed at Calsi-Crete, Inc., Pacific, Mo.



Company executives report that one man now handles the material which formerly required four men.

One of the four units currently in operation is used to remove aerated building material from steel molds which have been specially designed to work in conjunction with Vac-U-Lift. Other units are employed in planning and smoothing operations, for stacking and de-stacking, and for loading to road carriers.

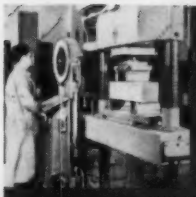
Material shown being lifted in above photograph weighs 165 lb; unit has a maximum gripping or lifting capacity of 600 lb; all units used are connected to overhead electric hoists.

Special brochure which explains the system and demonstrates its versatility is available.—Vac-U-Lift Co., Salem, Ill.

Concrete extrusion machine

An extrusion, slip-forming machine for the production of prestressed concrete has been introduced by Prestressed Equipment Co.

According to the company the unit will produce two rows of 8-in. keystone joist continuously at the rate of 3 ft per min or the equivalent of 360 lineal ft of joists per hr. It runs on a flat conventional type of prestressing bed 15 x 20 ft wide and 400 x 600 ft



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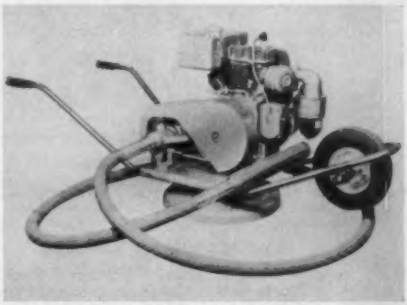
long; electrically operated, it obtains all its traction from the flat bed without the need of any lugs or special grippers cast into the bed. The machine is guided entirely by the prestressing strand used in the finished product.

By changing the form profile, several prestressed concrete shapes may be produced. The machine will be produced in two sizes. The small size machine will produce an 8-in. keystone joist, an 8-in. I-section joist, and an 8 in. deep T-joist; it will also produce a 4 in. thick slab flat. The larger machine will be able to take forms of various sizes to produce double t's, channels, T-joists, I-joists, and keystone sections having depths up through 20 in.—Prestressed Equipment Co., Lakeland, Fla.

High speed concrete vibrator

Empire's Model AKS concrete vibrator offers two sizes of vibrator heads of all steel construction, lifetime lubricated and sealed against moisture and dirt; eccentric element is mounted on double row ball



bearings; flexible cable is reversible and features a simplified plug-in design for attaching engine, vibrator head, or coupling. Heavy duty casing for cable has steel spiral liner enclosed by triple wire braid and abrasion resistant molded rubber outer casing.

Powered by a 5-hp Wisconsin engine with throttle control and centrifugal clutch, this model provides 9000 vibrating frequencies per minute in concrete; four shaft lengths available. Wheelbarrow carrying frame pivots with the engine thereby eliminating interference with the flexible shaft.—Empire Corp., 1566 S. 83rd St., Milwaukee 14, Wis.

Spray-on form release agent

Shell form compound, form lubricant for prestressed concrete beams and pipe, has been placed on the market by Shell Oil Co. Manufacturer states this form release agent permits quick stripping of concrete members, leaves concrete with a glossy, almost marble-like surface, and does not build up on the forms, making it suitable for architectural as well as structural grades.

The compound is a water-emulsion material mixed just before use in a ratio of 1 part compound to 5 parts water. Cost of the emulsion is about 20 cents per gal. Three gal. or less will cover a full 420 ft double-tee bed. Ordinary spraying equipment is adequate for application to steel and wood forms for all types of prestressed structural members. The compound is said not to

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interfere with concrete paints and coatings applied to architectural members. Address inquiries to Dan E. Hendricks, Jr.—Shell Oil Co., 50 West 50th St, New York 20, N. Y.

Adhesive for mounting polyethylene film

A fast-grab, high tack, waterproof adhesive, Bondmaster Z377, for mounting polyethylene film for permanent moisture control in building construction, airport runways, and similar applications has been announced by Rubber & Asbestos Corp. Sturdy bonds to concrete, wood, and a variety of other surfaces are claimed.

The material is a trowelable synthetic rubber-resin "pressure sensitive" adhesive that features quick "grab" upon contact with the polyethylene film up to 2 hr after the mastic is applied to the concrete. It can be applied to the surface of the wall, floor, ceiling, etc., with a stiff-bristled brush or a trowel. —Rubber & Asbestos Corp., Dept. P, 225 Belleville Ave., Bloomfield, N. J.

Pan forms for waffle ceiling

Designed to cut waffle ceiling costs, Acousti-Domes permit installation by simply positioning the domes, placing reinforcing rods, and casting slab, according to the manufacturer, thus eliminating rental of forms, placing and removal of pans, application of acoustical material, and roof slab insulation. The dome forms are 2 in. thick, made of perlite aggregate concrete, and measure 3 x 3 x 1 ft. Cast-in lifting lugs of wire are placed on opposite sides, enabling laborers to thread pipe through lugs for easy carrying.

The manufacturer also reports that Acousti-Dome ceilings are attractive and that the textured surface diffuses light without glare and soundproofs the room. —U. S. Construction Products Co., 2242 N. 35th St., Milwaukee, Wis.

Small concrete block machine

Slide-O-Matic concrete block machine features a power press action that exerts over 200 lb of pressure to make sharp, even-edged block in a smooth, easy operation according to a recent release by the manufacturer.

In further describing the unit the company states that it produces three 8 x 8 x 16-in. block per min; also produces many other sizes and types of concrete block such as patio block, concrete brick, bull nose, jamb and corner block. Quick change mold box cuts changeover time for different sized block to less than 10 min. Only one set of plain pallets of flat wood or steel is required.

Machine is operated by a 1-hp electric motor or 2-hp gasoline engine; it will produce 1400 block with one operator in an 8-hr day according to the company. —General Engines Co., Route 130, Thorofare, N. J.

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Seamless concrete slump cone

A seamless slump cone, designed and manufactured by Soiltest, Inc., is now available. The slump cone is made of heavy gage cadmium plated sheet steel and is of one-piece construction; handles and foot clamps are welded in position, also cadmium plated to resist rust. Made in form of a fustrum of a cone with a 8-in. diameter open base and a 4-in. diameter open top; height is 12 in.——Soiltest Inc., 4711 W. North Ave., Chicago 39, Ill.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

DESIGN AND SPECIFICATION OF WATERTIGHT MASONRY (OM-8A)—Six-page bulletin outlines six important considerations to be given in designing watertight masonry walls. Among other things the discussion covers the selection and proportioning of ingredients, types of mortar, compatibility of brick and mortar, importance of shrinkage control, and the role played by Omicron mortar-proofing in aiding both workmanship and watertightness.—The Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio

DORIX AGGREGATE FACING—Brochure describes Dorix, a thin, concrete, facing material with a finished surface exposing 85 or more percent aggregate. Lists properties of the five basic forms in which the facing is available, showing various applications and recommending proper installation methods. — Aggregate Surfaces, Inc., 644 N. Woodward Ave., Birmingham, Mich.

SPANG STANDARD UNDERFLOOR DUCT (Bulletin No. 484)—Illustrated 8-page bulletin presents Spang standard underfloor duct (formerly Howarduct) and its featured simplicity in distribution systems for electric power, telephone, and intercom. Catalog section includes detailed dimensions, various types and sizes of junction boxes, ducts, all accessories, and service fittings.—Spang-Chalfant Division of the National Supply Co., 2 Gateway Center, Pittsburgh, Pa.

STEELFORMS (Bulletin 300)—Twenty-page illustrated catalog on multi-purpose Formcrete steel forms for precasting reinforced or prestressed concrete. Semi-portable and interchangeable, with only four sets of basic I-beam steel forms, 30 different sizes of standard prestressed concrete I-beams can be produced. Custom and standard forms covered; specification charts included.—Food Machinery and Chemical Corp., Lakeland, Fla.

MODULAR BIBLIOGRAPHY—The Modular Building Standards Association has announced publication of a bibliography listing books, articles, and technical papers dealing with the modular system of coordinated dimensioning for buildings and products used in their construction.—Modular Building Standards Association, 2029 K Street, N. W., Washington 6, D. C.

MODERNIZED CEMENT PLANT FACT FILE (ER-3241-2)—Reprinted from a technical report, 12-page fact file fully describes improved cement processing operations at a recently-modernized eastern Pennsylvania plant.—The Fuller Co., Catasauqua, Pa.

PRESTRESSED CONCRETE PLANTS IN THE UNITED STATES—Listed by states, gives firm, street address, and city. Compiled as of June 30, 1958. $1.00 per copy.—MODERN CONCRETE, 431 S. Dearborn St., Chicago 5, Ill.

MOREHOUSE RING DYNAMOMETER, BULLETIN 160—Construction, application and operation of ring dynamometers are discussed in 4-page, two color bulletin. Describes in detail the accuracy of the ring dynamometer as a force measuring instrument. Lists both general and dimensional specifications of standard capacity ring dynamometers.—Morehouse Machine Co., 1742 Sixth Ave., York, Pa.

A TOUR OF THE CALAVERAS CEMENT COMPANY PLANT—Sixteen-page brochure describing the cement manufacturing process in detail. Thirty photographs and a flow chart accompany a simple explanatory text which in effect gives the reader a "tour" of the company's San Andreas plant.—Calaveras Cement Co., 315 Montgomery St., San Francisco 4, Calif.

CECO STEELDOMES—Sixteen-page manual describing Steeldomes, rigid, deep-drawn, one-piece units for use in forming "waffle-type" two-way dome slab construction. Contains complete tabular data covering sizes available and voids created; includes detailed descriptions of erection procedure, drawings of typical arrangements, and illustrations showing erection.—Ceco Steel Products Corp., 5601 West 26th St., Chicago 50, Ill.

ACCESSORIES FOR MICROSCOPY—Catalog of accessories for the entire microscope field. Also institutional brochure describing services the company is equipped to render as consultant in fields of industrial light, x-ray, and electron microscopy.—Ernest F. Fullam, Inc., P. O. Box 444, Schenectady 1, N. Y.

THERMAL INSULATION OF VARIOUS WALLS —Lightweight Concrete Information Sheet, No. 4, furnishes convenient information on insulating values of various walls and their components. Tables provide calculated values of the over-all coefficient of heat transmission and total resistance for a number of different composite walls.—Expanded Shale Clay and Slate Institute, National Press Building, Washington, D. C.

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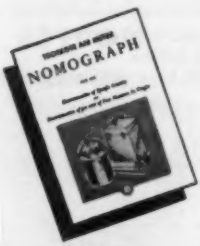

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DISCUSSION, PROCEEDINGS V. 54

Discussion of papers published in the January through March, 1958 JOURNALS appears in Part 2 of this September issue, as the Institute continues its quarterly publication of discussion. Since Part 2, September, is a part of *Proceedings* V. 54, discussion pages are numbered continuously from the final page of the June, 1958 JOURNAL. *Proceedings* V. 54 will be completed when papers published April through June are discussed in Part 2, December, 1958 along with index, errata, and contents for V. 54.

Listed below are the papers whose discussion is published in this JOURNAL.

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